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Contents

Foreword

The 10 papers in this Record were presented in two conference sessions at the 1989 Annual Meeting of the Transportation Research Board. Together they address a number of current concerns for aspects of sight distance design contained in *A Policy on Geometric Design of Highways and Streets* (AASHTO Green Book, 1984), including site-specific intersections, design vehicle variances, stopping sight distance, and passing sight distance. The papers document research supporting the need for a review and possible revision of those parts of the Green Book on criteria and standards for highway sight distance design.

Highway Sight Distance Design Issues: An Overview

JOHN C. GLENNON

This paper presents a brief overview of the conference session at which the papers in this Record were presented. The goal of this session was to present the best current thinking on the development and presentation of design criteria and values for stopping sight distance, passing sight distance, intersection sight distance, decision sight distance, and railroad-highway grade crossing sight distance. In looking at the adequacy of current design criteria and values, a major focus was the compatibility of these values with current and future operations of large trucks.

The papers in this Record were presented at a conference session held at the TRB Annual Meeting in Washington, D.C., on January 26, 1989. The session, "Highway Sight Distance Designs Issues," was cosponsored by two TRB committees: Geometric Design (A2A02) and Operational Effects of Geometrics (A3A08). Not only do these two committees have a long history of cosponsoring TRB sessions, workshops, and symposia concerning topics of timely interest, but they also have a longstanding focus on highway design policies in general and highway sight distance criteria in particular. This latter interest is manifested by the fact that seven of eleven participants in this outstanding conference session were members of these committees.

The seed for this session began when the members of TRB Committee A2A02 decided in 1986 to make sight distance its priority topic. At that time, the intention of the committee was to work toward a formal presentation of both the shortcomings of the current AASHTO Policy on Geometric Design for Highways and Streets (Green Book) and some clear recommendations for updating that publication (*1*). At about the same time, Committee A3A08 initiated planning for a midyear conference entitled "Beyond the Green Book." This conference, which focused on gaps in the Green Book and tools needed to supplement it, was held on November 9–10, 1987, in Austin, Texas (proceedings of the "Beyond the Green Book" conference are currently in press).

By far the major point of discussion that emerged from floor discussions and workshops at the midyear conference was the need to evaluate and update highway sight distance design values. Researchers pointed out not only the lack of evidence on safety effectiveness but also several inconsistencies of logic in the Green Book. Designers who attended the midyear conference were concerned with updated design values in the Green Book, particularly as they apply to reconstruction projects. In January 1988, therefore, Committee A2A02 decided to sponsor a session on sight distance at the 1989 Annual Meeting and was immediately joined in the effort by Committee A3A08.

John C. Glennon, Chartered. 8340 Mission Road, Suite B-12, Prairie Village, Kans. 66206.

IDENTIFICATION OF GREEN BOOK NEEDS

One of the major concerns about the Green Book is the level of commitment given to its production. This was the first major AASHTO design policy produced by AASHTO alone. Although inputs and critiques were solicited from selected segments of the industry, the major contributions were made by members of the AASHTO Task Force on Geometric Design, who individually prepared the chapters through generous donations of their time. Although these efforts are to be applauded, the question remains whether this level of effort represents an appropriate dedication of the highway community as a whole to a publication that directly affects the application of billions of dollars in highway funds. This author suggests that efforts were inadequate to make the Green Book an adequate reflection of the technology available at the time it was published. In particular, the following major needs have been identified by Committee A2A02 and should be addressed in future updates of the Green Book (many of these themes are evident in the papers presented in this Record).

Update Policies in More Timely Manner

The Green Book was the first major update of national highway design policies in 11 years for urban highways and in 19 years for rural highways. A more dynamic process should be adopted to ensure that policies are reviewed and updated in a more timely fashion. Although an update is under way at this writing, it appears that the revisions will be mostly cosmetic.

Involve Research Community

The Green Book failed to recognize major recent research findings both in the updating of policies and in the presentation of reference lists at the end of each chapter. The research community, through TRB and other appropriate groups, should be brought directly into the process to identify, critique, and evaluate research for inclusion in the design policies.

Develop Definitive Research Synthesis Program

A well-defined program of pragmatic research should be developed to synthesize methodologies to be incorporated into design policies that will promote optimization, design consistencies, and operational effectiveness.

Expand Design Criteria for Flexibility

A major shortcoming of all past design policies is that they have failed to acknowledge the need for design criteria structured to be sensitive to site-specific conditions, particularly with regard to the functional class of roadway. Many of the concerns about tort liability for design deficiencies could be abated by giving proper attention to developing design criteria that demonstrably reflect the operational and safety needs of the particular roadway. Detailed discussions of which design dimensions can be altered and which cannot, and how much they can be altered under differing conditions of traffic volume, composition, and speed; surrounding terrain and development; and functional class of roadway could go a long way toward the better use of highway funds. These discussions should of course include the tradeoffs among operational, safety, and economic goals.

Develop a Clearer Connection Between Design and Traffic Operational Criteria

In comparing the Green Book with the 1978 *Manual of Uniform Traffic Control Devices* (*2*), Committee A2A02 identified several major incompatibilities that are either ignored or rationalized away by these publications. The committee's position is that every design criterion should reflect the anticipated operation of the highway, including the compatible application of traffic control devices.

Stress the Importance of Highway Maintenance

The safe and efficient operation of highways depends on the continued maintenance of design factors such as cross-slope, superelevation, pavement skid resistance, sight distance, and related factors. The Green Book should emphasize the need to maintain these critical features.

Write Policies for Wider Audience

Current AASHTO policies, including the Green Book, are written almost entirely from the perspective of a state department of transportation. Broader consideration should be given to the wide variety of local jurisdictions that could use the policies as authoritative documents if the policies were properly focused.

EXAMPLES OF SIGHT DISTANCE DESIGN CONCERNS

The subject of highway sight distance offers an excellent illustration of the committees' concerns about design standards and the Green Book. Sight distance is one of the most basic design inputs affecting horizontal alinement, vertical alinement, and cross-sectional elements. Focusing on sight distance within the context of the Green Book thus represents a first meaningful step toward addressing the many concerns of both Committee A2A02 and Committee A3A08.

The two committees cosponsoring the 1989 conference session have identified several aspects of sight distance design that they believe need further consideration and development. Many of these concerns have been expressed above, several are repeated in the other papers in this Record, and a few are outlined further below.

Stopping Sight Distance

1. The Green Book model for stopping sight distance, which was adopted 50 years ago, is overly simplistic and clearly inappropriate for all highways under all operating conditions. Although the model inputs have been subjected to considerable fine tuning over the years, the model has scarcely been scrutinized for its relevance to the driving task. Valid concerns persist that the parameters used to exercise the model do not correspond to human visual limitations, to the likelihood that a given event will occur (for example, a 6-inch stationary object in the road), or to the ability to decelerate large vehicles. Several authors in this Record (Hall and Turner; Neuman; Urbanik et al.; and Harwood et al.) suggest more meaningful approaches to determining stopping sight distance design values.
2. The Green Book does not treat sight distance requirements relative to site-specific design or operational features (such as intersections).
3. The Green Book does not recognize possible design vehicle variances. For example, traffic data indicate that the eye height for a pickup truck driver may be more appropriate as the stopping sight distance for low-volume rural roads.
4. The Green Book stopping sight distance values are inconsistent for highway curves, not only because locked-wheel braking would cause loss of control, but also because the truck driver's eye height advantage is lost for sight obstructions such as walls or continuous vegetation (*3*).
5. Recent accident research, reported by Urbanik et al. in this Record, demonstrates some insensitivity of safety to Green Book design values. For example, the 6-inch stationary object does not represent a frequent or severe hazard. Also, highway sections with deficient sight distance (compared with Green Book values) do not necessarily show adverse accident experience.

Passing Sight Distance

1. The Green Book model incorrectly represents the sight distance need because it minimizes the possibility of aborting the pass (*4*).
2. A more appropriate operational model was first postulated in 1969, and several papers since that time have reiterated the flaws in the AASHTO model and presented usable alternatives (*4–9*).
3. Design and operational values given in the Green Book and in the *Manual on Uniform Traffic Control Devices* are incompatible (*4*, *5*, *7*).
4. The Green Book indirectly discourages any effective "design" for passing zones because it only considers overly long vertical curve lengths (Harwood et al. in this Record).
5. Recent use of the Green Book model has led several authors (*10–13*; Harwood et al., in this Record) to draw

flawed conclusions about the passing sight distance needs of large trucks.

Intersection Sight Distance

1. The Green Book presents Case I intersection sight distance as an alternative, then admits that it is not a safe practice. This flaw was first pointed out in the 1940 AASHO Policy (*14*).

2. The Green Book considers seeing the top inch of a passenger car as an adequate design object for intersection sight distance. This criterion is wholly inadequate for nighttime conditions where a vehicle cannot be seen above terrain obstructions until its headlights are visible.

3. The Case III-B sight distances are not practical, particularly if the design turning vehicle is a large truck (Mason et al., in this Record).

Railroad-Highway Grade Crossing Sight Distance

The minimum Green Book sight triangle actually promotes truck-train collisions because the truck cannot stop but can only clear from the sighting point (Fitzpatrick et al., in this Record). If the truck driver brakes, the truck will collide with the train before coming to a full stop. If the driver begins to stop and then decides to proceed, he will collide with the train before clearing the crossing.

CONCLUSIONS

Many issues persist concerning the development, presentation, adequacy, and usability of the AASHTO design values for highway sight distance design. This Record not only addresses a majority of these issues, but also provides different perspectives on many of the issues. The concerned reader should read all of the papers and all of the major cited references to see how they fit together to form a representative body of knowledge on the subject of highway sight distance design.

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Stopping Sight Distance: Can We See Where We Now Stand?

J. W. HALL AND D. S. TURNER

This paper examines the development of stopping sight distance (SSD) methodology over the past 75 years. Publications between 1914 and 1940 show that sight distance became increasingly important, but that it was not thoroughly understood. The emphasis during this period was on letting drivers see other vehicles in sufficient time to take evasive action. This concept changed drastically with the 1940 publication of AASHO's classic methodology, which made specific reference to objects, eye heights, and driver perception and reaction times. Evidence shows that the new procedures were gradually assimilated into the design process. Since 1940, emphasis has been on fine tuning the methodology by modifying its parameters. The paper discusses the prominent factors affecting SSD and traces their development over the past 75 years. The sensitivity of stopping sight distance to changes in the key parameters is examined. Characteristics and weaknesses of the methodology are discussed through a review of the recent technical literature. Five potential problems with the current AASHTO policies are discussed. Conclusions are drawn regarding the appropriateness of the current methodology and several specific recommendations are offered for additional research on this important topic.

Most highway and traffic engineers are familiar with the topic of stopping sight distance (SSD) as it is applied to the design and operation of streets and highways. They generally recognize that because of its dependence on human, vehicle, highway, and environmental factors, sight distance is a complicated issue. Although the publication of standards by AASHTO (*1*) might lead them to believe that this complex problem has been resolved, and that designs conforming to the standards will achieve the desired results, engineers are finding that it is expensive to comply with the current standards, especially in the reconstruction of existing highways. While this is a serious issue, with obvious financial and legal implications, its resolution may be hampered by our myopic view of the current sight distance standards. The intent of this paper is to examine the development of SSD methodology, to point out inconsistencies in the current procedures, and to pose topics for further research.

HISTORICAL DEVELOPMENT

Conventional wisdom has it that the origin of sight distance standards can be traced to a pair of 1940 AASHO publications (*2*, *3*). It would be inappropriate, however, to conclude that the issue was ignored at the state or national levels prior to that time. Early engineering textbooks, which typically emphasized the materials aspects of highway construction, devoted little attention to the subject. For example, the hazard of limited sight distance was recognized in a 1914 text (*4*, p. 97) by Blanchard:

> Sharp curves are points at which collisions are very liable to occur, particularly if the view is obstructed. Sometimes, if it is impossible to increase the radius of the curve, a great improvement can be obtained by clearing away obstructions so that the curve can be seen throughout its entire length when approached in any direction.

This was probably a good idea, but highway engineers of that era would argue that the suggested treatment was quite expensive. Two years later, coincident with the first Federal-aid Highway Act, a text (*5*, p. 45) by T. R. Agg advises:

> Safety Considerations. Steep grades, sharp curves and knolls that obstruct the view ahead should be avoided in the interest of safety. There should always be a clear view ahead for at least 250 ft and if a curve exists on a hill, the grade should be flattened around the curve if possible so as to permit a quick stop in case of emergency.

While Agg may have been correct, the engineer of today would feel uncomfortable with this statement. Why *250* ft? What about *speed*? What are the *object and eye heights*? And what kind of emergency is Agg talking about? There aren't any answers to these questions, because the statement above is the entire reference to sight distance in the text. But perhaps today's questions weren't appropriate then. Vehicles were taller and provided greater eye heights and ground clearance, while speeds were lower. By 1924, the third edition of Agg's text (*6*, p. 91) expanded on the issue of sight distance as follows:

> Curves. Horizontal and vertical curves, embankments, railroad crossings and intersections with other highways, constitute the dangerous portions of a public highway. . . . To minimize danger at curves it is desirable to provide ample sight distance and to construct horizontal curves with long radii and ample superelevation. The sight distance should be such as to permit a view of an approaching vehicle 400 ft away. That distance will permit both vehicles to be brought to a stop before colliding. Since the line of sight on a horizontal curve will depend upon whether the curve is in cut or not and upon the width of cleared right-of-way, no standard radius of curvature can be suggested that will provide the desired sight distance but it is easily computed in any case. . . . The radius of cur-

J. W. Hall, Department of Civil Engineering, University of New Mexico, Albuquerque, N. Mex. 87131. D. S. Turner, University of Alabama, Tuscaloosa, Ala. 35486

> vature for vertical curves that will give a sight distance of 400 ft is about 3,500 ft and this applies regardless of the rate of grade if the algebraic difference in grades exceeds about 5 percent.

Agg's revised text clears up the vagueness in the 1916 version by identifying what has to be seen and what driver action is expected, and indicating that the principle applies to both horizontal and vertical curves. The reference to the *radius* of a vertical curve is appropriate because some engineers in those days recommended the use of circular vertical curves to minimize earthwork. The distance has increased to 400 ft because of a better understanding of the issue, an increase in vehicle speeds, or both.

Early references to sight distance are not limited to textbooks; Brightman (*7*, p. 114) indicates that Michigan's practice in 1926 was to provide 500 ft of sight distance:

> The subject of visibility is one that cannot be overlooked and it is related to both horizontal and vertical alignment. In order that the motorist may always see far enough ahead for safety, the road should be so aligned that a clear vision of 500 feet ahead is available. This is worked out by long curves and the cutting away of banks which may hide the view in horizontal alignment. The vertical alignment is solved by the use of vertical curves in the grades which allow a car to be seen at a point 500 feet distant at all times.

Two years later, AASHO adopted a standard of practice on road design (*8*, pp. 12–13) requiring that "horizontal and vertical curves be used which provide a sight distance of at least 500 feet."

An informative article by Baldock (*9*, pp. 732–34) shows that in 1935 the practice in Oregon was to design all trunk highways, except through mountains, for vehicular speeds of 75 to 100 mph. As indicated by the following excerpts from Baldock's article, Oregon was cognizant of the problems in designing for these high speeds.

> Early in the consideration of vehicular speeds it was determined that three speeds would have to be considered as follows:
>
> 1. Critical speed, the maximum that can be attained with the standards used and beyond which only the most skillful racing drivers can operate without extreme hazards.
> 2. Designed speed, 80 percent of the critical speed and a speed that is safe for skillful drivers.
> 3. Recommended safe speed, which takes into account normal traffic conditions and the limitations of the ordinary driver. Hence a speed somewhat less than the designed speed.
>
> The critical speed of an automobile on a highway is controlled by the following factors: (1) the ability of the operator to function properly—the human equation, particularly in emergencies; (2) the ability of the mechanism of the vehicle to operate at high speeds without undue hazard; (3) the stopping distance or the distance travelled during the reaction time of the operators plus the braking distance; (4) the curvature . . . ; (5) the horizontal sight distance on curves, which, of course, varies with the curvature, the position of the vehicle on the road and the distance from the line of travel to the sight distance obstruction; (6) the sight distance over vertical curves; (7) the sight distance required in passing vehicles at varying speeds; and (8) the gradient used in the mountain sections.

Baldock clearly shows an understanding of the relationship between speed and sight distance. He also presents Oregon's method of calculating stopping sight distance, which is similar to current methods except that it assumes a driver reaction time of 0.5 sec. The frictional values used in the stopping distance equation were 0.5 on wet pavement and 0.8 on dry pavement; the latter condition is comparatively rare in Oregon. For a designed speed of 80 mph, the stopping distance would be 740 ft, and a sight distance of 1,500 ft is specified for vertical curves.

At the national level, there was increasing awareness of the relationship between design and safety. In 1935, for the first time, the Annual Report (*10*, p. 6) of the Bureau of Public Roads (BPR) included a brief section entitled Highway Safety, which states in part:

> In approval of plans for highway construction it [the BPR] has constantly endeavored to effect a desirable widening of surfaces, straightening of alignment and reduction of grades to make the roads suitable for the increased speed of modern traffic.

Nevertheless, the BPR report does not specifically mention the issue of sight distance. A text (*11*, p. 417) published in this same year offers the following guidance on sight distance:

> On double-track paved roads, the sight distance should be such that a driver can observe an approaching vehicle without being startled when travelling at normal road speeds and with the corresponding degree of concentration of attention given the road. On account of increasing automobile speeds a minimum sight distance of 600 ft is desirable.

Another indication of state practice in the mid-1930s is provided by a report (*12*) from the Ohio Department of Highways, which indicates that

> One danger point of the highway which receives more consideration than previously is the abrupt change at the crest of a hill where the driver is unable to see a safe distance ahead. On new construction and on reconstruction projects on main roads the sight distance on vertical curves are [*sic*] kept at a minimum of 1,000 feet on two-lane, 1,500 feet on three-lane, and 800 feet on four-lane pavements, unless this is economically prohibitive.

The Ohio report also presents a graph for determining the length of a vertical curve based on the algebraic difference in grades and the desired visibility length (500 to 1,500 ft). The chart is very similar to Figure III-39 in the current AASHTO standards (*1*) except that it assumes a 4.5 ft eye height.

Concern with sight distance was not limited to engineers in the United States. A British textbook (*13*) derives sight distance formulas for summit vertical curves that are similar to those used in current standards. The model assumes that two vehicles approach the summit from opposite directions, and the recommended sight distance allows each vehicle to decelerate to a stop before colliding with the other. The text notes that "one of the lowest cars on the road has an eye height of approximately 3 ft 9 in," and this figure is used in the calculations. The calculated braking distance of 240 ft for trunk roads with 60 mph speeds is doubled and then rounded to 550 ft "which gives the drivers half a second each to spare."

Standards for the 5,000-mi Reichsautobahn system, with a design speed of 112 mph (180 kph), provided for stopping sight distance. According to one source (*14*, pp. 292–94), the German calculations were based on a 1-sec perception-reaction time, and frictional coefficients of 0.4–0.5. Furthermore,

> In computing sight distance along curves and at summits a standard car is assumed, 4.9 ft high, with the eye of the driver 3.9 ft above the ground and 4.3 ft from the vertical plane of the two right wheels of the car. Two kinds of obstacles are considered: (1) A standard car and (2) an object near the summit projecting 20 cm (about 8 in) from the ground surface upwards.

The use of a lower eye height presumably reflected the design of the vehicle fleet. The standards recognized the importance of considering objects other than vehicles. In level terrain, their designs often used horizontal curve radii greater than the 6,560 ft necessitated by these standards; however, design standards were lowered when topography made compliance uneconomical.

The BPR's 1936 Annual Report (*15*, p. 15) specifically recognized the importance of providing adequate sight distance in the interest of enhancing highway safety.

> One matter that confronts highway officials which is of great present importance and which will be of much concern in the future is the eradication of those conditions that are now or may be conducive to accident, injury, and death. . . . The greatly increased speed of motor-vehicle travel requires a general increase in sight distances and the elimination of obstructions to view at intersections.

The literature of the day seemed to have difficulty in distinguishing among the various reasons for providing adequate sight distance. One author (*16*, pp. 21–24), however, appears to concisely identify two principal types of sight distance:

> The general feeling is that 1,000 ft is the shortest sight distance that may be regarded as reasonably safe on two-lane roads to be traveled at 60 miles per hour. In a distance of 1,000 ft a driver of a vehicle moving at 60 miles per hour can normally pass another vehicle moving at 40 miles per hour and avoid collision with an approaching vehicle traveling at 60 miles per hour.
>
> Sight distance on four-lane highways having parkways or median strips separating the opposing traffic lanes may be reduced appreciably below those on two-lane or three-lane roads because the possibility of accidents on them is limited largely to rear-end collisions. The safe sight distance for four-lane roads, therefore, should be at least that distance in which a moving car can be brought to a full stop. It will range from 300 to 700 feet depending on the speed and braking power of the vehicles involved. The American Association of State Highway Officials recommends a minimum sight distance of 500 feet for four-lane roads and 800 feet minimum on other Class A and B roads.

Elsewhere, the same article notes that AASHO *recommends* a minimum sight distance of 800 ft on horizontal curves; a reduction to 500 or 600 ft for design speeds of about 40 mph is permitted in mountainous terrain. This recommendation and the previously cited standard of practice (*8*) clearly show that AASHO was providing guidance on sight distance issues prior to the publication of its 1940 policy (*2*).

The first discussion on this topic by the Highway Research Board was in 1937, when the Proceedings (*17*) included a report from its Committee on Sight Distances. The report introduced the concept of nonpassing sight distance and identified several areas where additional research was needed to properly quantify the parameters involved. Several of the report's most significant conclusions are as follows:

> If safety is to be built into our highways, it is vitally necessary that the road be opened up to view for a sufficient distance to enable the driver . . . to control the speed of the vehicle to avoid encountering unexpected obstacles in its path [p. 111].
>
> The assumed design speed of a highway is considered to be 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. . . . The length of highway visible to a driver at every point should be in excess of the distance required to bring the vehicle to a stop before reaching a stationary object in the same lane when travelling at the assumed design speed of the highway. This distance may be termed the safe stopping distance. Values for the factors entering into its determination should be chosen conservatively in order that drivers who normally drive faster than the assumed design speed and drivers who do so occasionally also may avoid encountering obstacles in the road [p. 112].
>
> For non-passing minimum sight distance two seconds for perception time, one second for brake reaction time, and 0.4 for the uniform coefficient of friction may be considered reasonable values. They result in non-passing minimum sight distances equal in feet to about ten times the assumed design speed in miles per hour. The variation is not uniform, being greater at high speeds and less at lower speeds. For four-lane and divided highways a greater margin of safety may be advisable. This may be secured by assuming a speed 10 miles per hour greater than the assumed design speed of the highway for sight distance purposes [p. 118].

Despite the extensive discussion of sight distances in the HRB paper, there is no mention of eye or object heights. For rural highways with a design speed of 60 mph, the stopping sight distance calculated using these HRB procedures is about 11 percent less than current AASHTO standards. But, if one accepts the admonition to assume a speed that is 10 mph greater than the actual design speed, the calculated sight distance is 13 percent greater than current AASHTO values.

The BPR's 1937 report (*18*, p. 2) again recognized the importance of sight distance and implied that a substantial portion of the highway system posed a hazard to motorists.

> Construction of through routes was begun some 15 or 20 years ago when the speed of vehicles was much slower and traffic considerably less in volume. The roads built were designed for conditions as they were then foreseen, and were influenced somewhat by the necessity of rapidly extending the mileage. Engineering standards in respect to sight distance, curvature, and grade have been steadily raised but much of the early construction reflects the earlier standards and is unsafe for modern traffic. . . . The condition of these highways cannot be considered satisfactory so long as many sections present unexpected dangers to the motorist.

In a section of its 1938 Annual Report (*19*, pp. 2–4) entitled Greatest Needs on Main Roads are Widening, Longer Sight Distances, and Reduction of Curvature, the BPR stated:

> Eliminating those curves that have become traffic hazards at the now normal driving speed and increasing sight distances by road straightening and by grading at the tops of hills are widespread needs on the existing main highways. These defects are found generally on roads in every part of the country and their danger to traffic is the consequence of an increase in vehicle speed far beyond what was visioned 15 or 20 years ago and far in excess of the legal limitations that existed in most states.

In the same report, however, the BPR blames drivers for accidents on roads it describes as hazardous. In a year when 32,000 persons died on the nation's highways, BPR reports:

> At the same time it must be recognized that accomplishment of all these things (e.g., sight distance improvements) will not constitute a solution of the accident problem. The present condition of main highways is not conducive to accidents except when rendered so by risk taking drivers. The data available on the causes of accidents indicate that improper acts by vehicle drivers are the element common to most accidents. . . . There is a relatively small group of definitely accident-prone drivers who experience a relatively large number of accidents.

A 1939 textbook (*20*) notes that 5 ft is generally used as the height of the driver's eyes above the road surface. With respect to vertical curves, the text notes:

> The minimum length of vertical curves at summits will be governed by the distance within which two vehicles are within sight of each other, this distance being defined as the "sight distance." Proper sight distance varies for different conditions and should be greatest on roads having smooth pavements with no parking strips between opposing lanes of traffic. Recommended sight distances vary from 350 to 1,000 ft for rural highways, the maximum being applicable to high-speed through highways.

For horizontal curves, this text recommends minimum sight distances of 800 ft (measured along the roadway center line) on primary and heavily traveled secondary state highways, although 500 ft is a desirable minimum on local highways. The text also provides the following tabulation of stopping distances:

	Stopping Distance, Feet	
Speed	*Quick Thinker, Good Brakes*	*Slow Thinker, Fair Brakes*
20	30	55
40	100	170
60	200	330
80	325	550

Finally, in 1940, Agg published the fifth edition (*21*, p. 154) of his text; his discussion of sight distance at vertical curves suggests that the concern is drivers seeing each other.

> When two vehicles approach the top of a hill from opposite directions on a highway at least two lanes wide, there is no element of danger if each is held to its proper lane and the drivers are able to see each other while they are still a reasonable distance apart. The line of sight of an automobile driver is about 5 ft above the road surface. With that factor fixed, the curvature is readily computed for any desired sight distance. The problem then becomes one of determining what constitutes a reasonable sight distance, but upon this point it is not easy to be specific. Perhaps a good basis for preliminary computations is to determine how much distance is required to bring a vehicle to a stop from the extreme road speed to be expected (if there is any such thing as a limit to speed, which seems doubtful). If the road surface permits a reasonable application of the brakes without starting a skid, a vehicle with four-wheel brakes could be stopped in about 300 ft from a speed of 60 mph. To this must be added about 75 feet as the distance traveled during the "reaction time." This would indicate that about 800 feet is the minimum sight distance for summits on busy trunk-line highways. Many of the state highway departments are designing the trunk highways with a sight distance of 1000 ft or more.

This overview of sight distance prior to 1940 suggests that the issue was recognized as being important, but it was not thoroughly understood. The emphasis was to provide sufficient sight distance for the driver to see other vehicles in sufficient time to avoid them. In only one domestic reference (*17*) is the concept of avoiding obstacles in the road discussed. Only the foreign authors (*13*, *14*) used an eye height of less than 4.5 ft. In hindsight, AASHO's 1940 sight distance policy (*2*) represented a significant change from previous practice, although interestingly enough it received no notice in *Engineering News-Record*, which covered most of the highway developments of that era. The minimum sight distances based on the revised procedures ranged from 200 ft at 30 mph to 600 ft at 70 mph. Some highways designed according to previous criteria did not satisfy the new requirements.

Perhaps the most dramatic change introduced by the 1940 standards was the substitution of a small object as the feature that must be seen. AASHO's selection of a 4-inch-high object (*2*) was justified as follows:

> The stationary object may be a vehicle or some other high object, but it may be a very low object such as merchandise dropped from a truck or small rocks from side cuts. To be on the safe side the surface of the pavement should be visible to the driver for the entire length of the non-passing sight distance, but the necessity for it is questionable. Large holes rarely are encountered in modern pavements and very small objects generally can be avoided without the necessity for stopping. Therefore, a height of object of 4 inches is assumed in determining non-passing sight distance.

Table 1 summarizes the historical development, the 1940 standards, and the well-documented changes over the past 48 years. The 1940 policy established the fundamental methodology that is still in use today, but there has been a fairly continuous change in individual parameters toward safer values. For example, driver eye height has been reduced by a foot, whereas pavement friction values were reduced to approximately 70 percent of their original values. The only element that has not changed significantly is the driver; as a result, the assumed perception-reaction time has remained virtually constant.

REVIEW OF SELECTED RECENT RESEARCH

Glennon (*26*) performed a critical review of SSD literature for a Transportation Research Board report to Congress. He drew the following (paraphrased) conclusions:

1. Alignment changes performed to improve stopping sight distance appear to be safety-effective when very short portions of a roadway are improved to provide very long sight distances.
2. Alignment changes are normally cost-effective only on highways that have (a) very high traffic volumes and (b) major hazards that are hidden by a sight obstruction.
3. Highway agencies must be careful when making minor lengthening of extremely substandard crest vertical curves. Unless care is used, it is possible to provide better sight distance for a short length of highway while causing an increase in the total length of roadway with inadequate sight distance.

TABLE 1 HISTORY OF STOPPING SIGHT DISTANCE PARAMETERS

Source	Reference No.	Pavement Condition	Stop Condition	Eye Height (ft)	Object Height	Friction Factors	Speed	Perception-Reaction Time (sec)	Sight Distance (ft)
Agg, 1916	5								At least 250
Harger, 1921	22			5.5	5.5 ft				
Agg, 1924	6				~5 ft				400
Michigan, 1926	7				~5 ft				500
Oregon, 1935	9	Wet, dry			~5 ft	0.5 wet 0.8 dry	80% of critical speed	0.5	1,500 @ 80 mph
Wiley, 1935	11				~5 ft		Normal road speeds		600
Ohio, 1937	12								1,000 (two lanes) 800 (four lanes)
Conner, 1937	16				~5 ft				500 (four lanes)
HRB, 1937	17				Unexpected obstacles	0.4	Maximum uniform speed	3	
Bateman, 1939	20			5	~5 ft				800 (Horiz C)
Agg, 1940	21			5	~5 ft	0.40 @ 60 mph		<1	
AASHO, 1940	2	Dry	Locked wheel	4.5	4 in.	0.50 @ 30 mph 0.40 @ 70 mph	Design speed	3 @ 30 mph 2 @ 70 mph	200 @ 30 mph 600 @ 70 mph
AASHO, 1954	23	Wet	Locked wheel	4.5	4 in.	0.36 @ 30 mph 0.29 @ 70 mph	85–95% of design speed	2.5	
AASHO, 1965	24	Wet	Locked wheel	3.75	6 in.	0.36 @ 30 mph 0.27 @ 70 mph	80–93% of design speed	2.5	
AASHTO, 1970	25	Wet	Locked wheel	3.75	6 in.	0.35 @ 30 mph 0.27 @ 70 mph	Min: 80–93% of design speed	2.5	
AASHTO, 1984	1	Wet	Locked wheel	3.5	6 in.	0.35 @ 30 mph 0.28 @ 70 mph	Design speed	2.5	200 @ 30 mph 850 @ 70 mph
NCHRP, 1984	29	Wet	Controlled	3.33	4 in.	By numerical integration	Design speed	2.5	

4. 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. The *Limited Sight Distance* sign has been ineffective in providing the proper warning to motorists.

5. Stopping sight distance on horizontal curves may be a particular problem. Cornering forces on the tires consume a portion of the friction force that might otherwise be used for deceleration. In addition, large trucks require longer SSDs than cars. For vertical curves, the truck driver's increased eye height offsets the required additional stopping distance; this advantage is not available for horizontal curves.

6. Low-cost treatments such as clearing vegetation or removing other minor obstacles on the inside of horizontal curves is a cost-effective technique to increase SSD on virtually all highways. Minor clearing on the inside of a curve can sometimes produce spectacular increases in SSD.

7. The skid resistance of pavement on the approaches to a limited sight distance roadway section might receive particular consideration.

Neuman et al. (*27*) examined the functional requirements of stopping sight distance. Their study identified several inconsistencies in the present AASHTO policy, including the following:

1. SSD accidents are event-oriented. The mere presence of a segment of highway with inadequate SSD does not guarantee that accidents will occur. Inadequate SSD is simply one item in a chain of events that leads to a collision. For example, a site with a minimally limited SSD on a low-volume, low-speed, rural route would rarely produce a critical linkage of events to cause a collision.

2. The probabilities of occurrence of SSD-related critical events define the relative hazard of any individual location. The joint probability of an accident is a function of traffic volumes, frequency of conflicts, and other factors.

3. The severity of SSD-related collisions may be more important than the frequency of such accidents. High-severity collisions may dominate cost-effectiveness studies of potential improvements.

4. There are many minor, uncontrollable factors that contribute to accidents at limited SSD locations. These minor factors (worn tire tread, deficient vehicle braking characteristics, irregular pavement, impaired driver, and the like) become more important when the driver enters a critical situation and tries to avoid an accident.

These researchers also report that at locations where deficient sight distance is caused by short vertical curves, lengthening the vertical curves could make the situation worse. Although the degree of SSD deficiency, as reflected by the safe speed, may be improved, the distance over which a driver experiences a deficiency may increase. In other words, an expensive reconstruction project might transform a short vertical curve with a seriously restricted sight distance into a longer vertical curve with only a marginally higher safe speed.

Neuman and Glennon (*28*), in an effort hampered by the lack of data, attempted to establish the cost-effectiveness of SSD improvements. They were able to establish upper limits on the effectiveness of sight distance improvements by constructing a model based on optimistic assumptions. Their model showed quite clearly that eliminating SSD deficiencies by making geometric changes to vertical or horizontal curves could only be justified in the presence of high traffic volumes or when significant hazards existed within the restricted sight area.

Olson et al. (*29*) performed a series of controlled roadway experiments to evaluate perception-reaction time, driver eye height, object height, and braking distances. Their findings caused a significant stir in the highway engineering community. Although they found that the 90th-percentile test driver had a perception-reaction time of 2.4 sec, they recommended retention of the traditional 2.5 sec because their test drivers probably had a heightened awareness and were not subject to factors (for example, fatigue) faced by normal drivers.

On the basis of their results, the researchers recommended changes to parameters in the AASHTO SSD equations. They proposed that the driver's eye height be reduced from 42 to 40 in, and that the object height be reduced from 6 to 4 in. In addition, they suggested another deviation from current AASHTO procedures. They contended that a driver will "modulate his braking control" so that he can decelerate without losing directional stability. They recommended a numerical integration technique for calculating braking distance, rather than relying on the AASHTO locked-wheel, skid-to-a-stop method.

The researchers also concluded that the higher eye height of truck drivers allowed them to initiate braking sooner at locations where sight distance is restricted by vertical curvature. As a result, stopping sight distance requirements for large trucks under these conditions are reasonably similar to those for passenger cars. Other researchers (Harwood et al., in this Record) report that this may not be the case because of variances in truck drivers' braking skills. In addition, the increased truck driver eye height provides no benefit when emergency stopping conditions exist within sharp horizontal curvature.

MEASURING THE RELATIONSHIP BETWEEN ACCIDENTS AND STOPPING SIGHT DISTANCE

The primary reason for increasing SSD is to provide improved safety benefits to motorists. Unfortunately, it is difficult to know how much to improve SSD to obtain a given level of improvement because data are lacking on the relationship between SSD and accidents. Several studies have examined sight distance as one of many factors contributing to crashes; nevertheless, these general research efforts have failed to produce a realistic model to define the change in accident rates for specific treatments that change sight distance.

Two studies offer possible insight into the issue. Farber (*30*) employed a Monte Carlo simulation technique to investigate accident potential for a limited SSD situation. He investigated the hypothetical situation of left-turning vehicles just downstream from a sight-distance-limiting crest vertical curve. He was able to draw conclusions about accident potential as a function of traffic volume, sight distance, and related factors. Such simulation methods have previously been used in other types of research to gather realistic data on parameters such as conflicts, operating conditions, and accident potential. Continued development of Farber's model might be a useful way to develop a similar model describing the relationship between safety and stopping sight distance.

Olson et al. (*29*) performed a statistical analysis on ten pairs of sites that were matched for similarity—except for their sight distance. In seven of these pairs, the limited sight distance site had more accidents than its companion. In two cases, the limited and full sight distance pairs had approximately the same number of collisions, whereas in one case, the limited sight distance location actually had fewer accidents than its matching site. Overall, the limited SSD sites had a 50 percent higher accident rate than the locations with adequate SSD.

With the exception of these two studies, there does not appear to be any work that conclusively defines the relationship between limited SSD and accident rates. The absence of sufficient data on this issue limits our ability to predict the results of changes to the existing methodology.

SENSITIVITY OF STOPPING SIGHT DISTANCE PARAMETERS

The methodology for calculating SSD is found in the AASHTO Green Book (*1*). The basic equations to determine SSD at crest vertical curves utilize six variables:

1. Perception-reaction time,
2. Driver eye height,
3. Object height,
4. Vehicle operating speed,
5. Coefficient of pavement friction, and
6. Algebraic difference in grades.

If the basic SSD methodology is to be modified to improve roadway safety, an understanding of the role and sensitivity of each of these parameters is necessary. In other words, if any one of the parameters is to be changed, it is important to know the effect on other parameters and the resulting overall change in sight distance. Five of the parameters will be reviewed in the following discussion; the sixth parameter, algebraic difference in grades, is a product of local site conditions and is not specifically discussed. Because several of the references dealing with eye and object height changes also report the effect on crest vertical curve length, this latter characteristic is included in the comparisons.

Perception-Reaction Time

Recent research has confirmed that 2.5 sec is a reasonable perception-reaction time regardless of design speed (*29*). Woods (*30*) noted that any change in perception-reaction time was actually a change in the distance traveled at the design speed. Thus, the effect on SSD was highly speed dependent. Glennon (*32*) and Farber (*33*) indicated that for changes in perception-reaction time, the increase in SSD became significant at higher speeds. Hooper and McGee (*34*) reached the opposite conclusion, contending that at higher speeds the braking component of stopping sight distance became the dominant factor, even though a significant distance was traveled during the increased perception-reaction time.

Driver Eye Height

The sensitivity of eye height appears to have been thoroughly investigated and reported in the technical literature. As shown in Table 2, stopping sight distance has been found to be relatively insensitive to changes in driver eye height. The data

TABLE 2 EFFECT OF CHANGES IN EYE HEIGHT ON STOPPING SIGHT DISTANCE AND VERTICAL CURVE LENGTH

	Source					
	Farber (*33*)	Olson (*29*)	Khasnabis (*35*)	AASHTO (*1*)	Woods (*30*)	Woods (*30*)
ΔEye height (in.)	−6	−2	−3	−3	−1.2	−6
Percent change	(−15)	(−5)	(−7)	(−7)	(−3)	(−13)
ΔSSD (%)	+5	+1.5	+2.7	+2.5	—	—
ΔCurve length (%)	—	+3	+5.3	+5.0	+2.3	+11.5

TABLE 3 EFFECT OF CHANGES IN OBJECT HEIGHT ON VERTICAL CURVE LENGTH

	Source					
	Khasnabis (*36*)	AASHTO (*1*)	Woods (*32*)	Khasnabis (*36*)	Olson (*29*)	Woods (*32*)
ΔObject height (in.)	6 to 0	6 to 0	6 to 2	6 to 3	6 to 3	6 to 4
Percent change	(−100)	(−100)	(−67)	(−50)	(−50)	(−33)
ΔCurve length (%)	+79	+85	+24–30	+18	+10	+12–16

in the Table regarding vertical curve length changes are based on the specific set of assumptions employed by the researchers, so an absolute comparison is not possible.

Even with the inherent differences between the studies, Table 2 still shows that a 3-in decrease in eye height only produces a 3 percent increase in the necessary SSD, while a 6-in decrease requires a 5 percent increase in SSD. There is a relatively strong consensus among these researchers that a moderate reduction in driver eye height produces a small resultant change in vertical curve length and stopping sight distance.

Object Height

Considerable research has been devoted to the role of object height in determining SSD. Four studies, summarized in Table 3, indicate that stopping sight distance is more sensitive to object height than to driver eye height. For example, moving from a 6-in object to a 0-in object increases crest vertical curve length by approximately 80 percent. Smaller reductions in object height have a less drastic effect; halving the object height to 3 in increases the vertical curve length by 10–18 percent, depending on assumptions.

Vehicle Speed

At least three investigators have determined that travel speed is an extremely sensitive parameter. Woods (*30*) showed that a 10 percent increase in vehicle operating speed yielded an increase of approximately 40 percent in crest vertical curve length for speeds between 40 and 65 mph. Farber (*33*) found that "small deviations in speed are equivalent to large deviations in stopping sight distance." This same conclusion is supported by others (*35*).

Pavement Friction

The most sensitive parameter in determining SSD appears to be the pavement friction value. Farber (*33*) found a noticeable relationship between friction effects and design travel speed, and he observed that SSD sensitivity to pavement friction increased with speed. Woods (*30*) states that "pavement friction is the most sensitive parameter in crest vertical curve design." He showed that for f values near 0.35 (a fairly low value, comparable to the 15th percentile of a typical state's total friction measurements), crest vertical curve length increased about 4 percent for each 0.01 change in pavement friction. As friction values dropped lower, curve lengths increased at an even greater rate. For very low f values (around 0.10), a change of only 0.01 caused a 20 percent change in vertical curve length.

The highest level of sensitivity is at the lower end of the pavement friction scale. This is also the region in which braking characteristics are poorest. Thus, at locations where a high degree of hazard already exists because of low f values and marginal sight distance, relatively minor changes in f produce drastic changes in SSD. The worst possible effect occurs at the worst possible location.

Pavement friction values are partially dependent on environmental conditions. Hill and Henry (*36*) report that a pavement's f value will decrease by more than 0.01 for a temperature increase of 10°C. In some parts of the country, daily temperature swings are twice this much, resulting in friction changes of 0.02–0.03. Based on the 15th-percentile f value of 0.35 reported by Woods, this change in temperature could increase the necessary SSD 8 to 12 percent.

Summary of Parameter Sensitivities

Of the five parameters reviewed, the most sensitive is pavement friction, followed by vehicle operating speed. The least sensitive appears to be driver eye height. It is interesting, however, that most recent research and discussions have focused on driver eye and object heights, two of the less sensitive parameters in determining stopping sight distance. The potential daily change in ambient temperature at sites with low friction values has a significant affect on SSD, yet the issue has received relatively little attention.

POTENTIAL PROBLEMS WITH THE CURRENT METHODOLOGY

A review of the historical development of SSD demonstrates that the early highway engineers did not have a thorough understanding of the issue. In 1940, AASHO set up a plausible model with the potential to at least standardize this design parameter. Efforts during the past half-century have focused on adjustments to this model, although comparatively little attention has been directed to its validity. From a macroscopic view, the basic model possesses certain difficulties that warrant further attention.

Driver Vision Requirements

Considerable attention has been directed to the subject of object height, and in the last decade it has been reported that the undercarriages of many passenger vehicles cannot clear an object 6 in high. This has led to suggestions that the object height should be lowered. A reduction, however, may be unrealistic. Consider the situation where 600 ft of sight distance is required. The current model assumes that on a tangent, level road, the normal driver should not have a problem seeing a 6-in-high object at this distance; in the absence of atmospheric interference, this corresponds to seeing a 0.2-in-high object at 20 ft. By comparison, the standard letters on a 20/20 eye chart are 0.35 in high, whereas the 20/40 letters are 0.70 in 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.

Furthermore, in the case of a vertical curve, the driver is faced with an additional problem: the entire object (either 6 or 4 in high) doesn't suddenly become visible. Rather, the driver initially has a line of sight to the very top of the object; as he approaches the object, there comes a point where he has a line of sight to the bottom of the object. Consider a 1,600-ft vertical curve with an algebraic difference in grades of 5.9 percent; with this design, a driver with an eye height of 3.5 ft will have a line of sight to the top of a 6-in-high object at a distance of 600 ft. Nevertheless, the driver will not have a line of sight to the entire object until he has approached to within 435 ft. The separation between where the driver might first see the top and the bottom of the object corresponds to approximately 1.9 sec at 60 mph, or 75 percent of the assumed conservative perception-reaction time.

Probability of an Accident with a 6-Inch-High Object

The previous section made the point that a typical driver will have difficulty in seeing a 6-in-high object at rural highway speeds. But it is also appropriate to consider how frequently an object of this kind is actually struck in an accident. Analysis by Woods (*30*) found that the collision rate for objects of this size or smaller was only 0.02 per million vehicle miles. This is at least two orders of magnitude smaller than the collision between pairs of multiple vehicles. The small probability of a collision with objects of this size suggests that we may be designing for an event that almost never occurs. In addition, a change to a 0-in object height, however desirable from a theoretical viewpoint, would appear to have a negligible effect on accident rates; it is questionable whether drivers can discern such small objects at the distances required for rural highway speeds.

Liability Trends

State highway agencies paid an estimated $120 million in judgments and settlements from tort liability claims in 1986, and spent at least another $20 million in defending these cases [Turner et al. (*37*)]. Engineers are properly concerned about this issue, especially since the number of suits is growing at an annual rate of 17 percent. Data are not readily available to show what share of these suits involve contentions of inadequate stopping sight distance, but the previously cited accident data imply that the number would be small. Although it would take additional research to reach a definitive conclusion, it appears that the extensive financial resources required to upgrade older roads to current SSD standards might be better spent on alternative improvements.

Vehicle Characteristics

There is no doubt that the current methodology does not provide adequate SSD for trucks on horizontal curves, regardless of object size (*26*). A current study (*30*) has found that a truck driver's ability and efficiency are major factors in assessing whether current standards are sufficient for large trucks in individual SSD maneuvers. Potential changes in braking systems might reduce the disparity between trucks and passenger vehicles, but as with any change to the vehicle fleet, this would be a longer-term solution. In the meantime, truck accident experience related to SSD warrants further examination.

Another vehicle characteristic, the lighting system, has not been given proper attention in the development of the SSD model. Previous discussion has noted that the driver may have difficulty detecting a 6-in-high object during the daylight at highway speeds. With properly aligned low-beam headlights, the driver on a typical rural road at nighttime will not be able to see an object in the road at these same speeds.

Pavement Friction

AASHTO (*1*) describes the frictional coefficients used in the model as *generally conservative*. Although there is a general consensus that designing for adequate stopping sight distance on an icy road is inappropriate, it is proper to recognize that pavement friction can change significantly in response to environmental factors. The variation of friction with increasing temperature was previously noted. In addition, the frequency and intensity of rainfall that serves to cleanse the pavement, as well as the quality of materials used in the pavement, can have a significant effect. Even if the SSD model had no other faults, its application to a specific location using an assumed nearly all-inclusive frictional coefficient may produce a substandard design.

CONCLUSIONS

Prior to the 1937 report by the HRB Committee on Sight Distances, the rationale for a policy on stopping sight distance was poorly understood. Emphasis was placed on providing sufficient distance for a driver to see and avoid other vehicles, but the distances were not analytically related to driver, vehicle, or roadway characteristics. Roadbeds and roadsides designed in compliance with these minimum recommended sight distances became substandard as vehicle speeds continued to increase. The methodology described in the 1940 AASHO policy sought to incorporate the factors that influence a driver's ability to respond to obstacles in the roadway. Since that time, the methodology has remained unchanged, although the individual parameters have been fine-tuned in an effort to account for changes in roads, vehicles, and driver behavior.

Stopping sight distance has become a topical issue for several reasons. Designers argue, for example, that recent adjustments to individual parameters in the model have had two effects:

1. Highways designed in accord with previous policies have suddenly become substandard, thus creating potential liability problems.
2. The expense of meeting the revised standards in the construction of new highways, and especially in the reconstruction of older alignments, adds significantly to project cost. This issue is critical because the benefits of the revisions have not been demonstrated.

If the highway engineering community had faith that the current SSD model reflected the needs in actual driving conditions, the foregoing effects could probably be accepted. There is growing concern that the 1940 model does not, and perhaps cannot, reflect the realities of driving. On one hand, it does not properly account for driver vision limitations, large trucks, nighttime driving, and realistic variations in pavement friction. On the other hand, further model adjustments to resolve these theoretical shortcomings may not be justified because

1. Available accident data fail to support the contention that the type of incident that the SSD model is intended to guard against is even a minor problem on existing rural highways.
2. The significant extra costs of highway construction and reconstruction occasioned by adjusting model parameters to reflect the extremes of current or projected driver, vehicle, or highway conditions could prove detrimental to overall highway system safety if the limited funds for improvement are used to provide an optimal, rather than a realistic, level of highway safety.

RECOMMENDATIONS

There is clearly a need to reexamine the role and importance of stopping sight distance in the safe operation of streets and highways. Although several ongoing and recently completed studies have examined individual components of this issue, and have made valuable contributions to the state of the art, there is a need for a more thorough study that would address the following issues:

1. Does the current model for stopping sight distance address a real problem, as exemplified by actual accident experience on sections of road that do not meet current standards, or is it a theoretical aberration that does not properly reflect actual operating conditions?
   a. If the model properly portrays realistic hazards on the highway system, what, if any, modifications are needed to better accommodate these conditions?
   b. If the model does not adequately represent the conditions experienced by the average, reasonably prudent driver, what methodology is required to reflect realistic conditions?
2. Since compliance with current SSD standards on reconstructed highway segments limits the number of projects that can be undertaken within budgetary constraints, what is the systemwide tradeoff among SSD, highway safety, safe roadsides, and other design and operational factors that influence safe roadway operation?
3. Other transient hazards on the roadway, most notably animals but also stalled vehicles in a traffic lane, create hazards for the motorist. Has too much attention been devoted to the *theoretical* 6-in-high object in the roadway?
4. Although AASHTO standards are developed and accepted by state highway agencies, and are applicable to rural highways under their control, they are often imposed in a *de facto* manner on local roads administered by counties and cities. There is a need to establish the relevancy of AASHTO's design standards in general, and SSD standards in particular, to local roads.

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New Approach to Design for Stopping Sight Distance

TIMOTHY R. NEUMAN

Design for stopping sight distance (SSD) is among the most basic, critical considerations in the total design of a highway. SSD requirements affect all geometric elements—horizontal and vertical alinement and cross section. Despite the importance of SSD, there is continuing, growing dissatisfaction among many design engineers with the current policy and general approach toward SSD. Such dissatisfaction can be attributed to the problems and costs of meeting current design policy, which have changed in recent years, coupled with a lack of evidence of the safety effectiveness of the policy. This paper presents a new approach to SSD design. It involves the abandonment of the concept that a single operational model for SSD is appropriate for all highway types under all conditions. Instead, the approach presented here suggests functional highway classification as the foundation for determining SSD design policy and values. A range of different operational models and driver, vehicle, and roadway parameters would be possible for different classes of highways. This, in turn, allows a range of design values for SSD for a given design speed, rather than just one value for all conditions. The paper presents examples of such models, with assumed values for driver reaction time, pavement friction, and object height. Illustrative calculations of SSD for five different classes of highways are shown. The calculations indicate the potential for SSD design values to vary significantly from those currently shown by the AASHTO policy.

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Despite the importance of SSD, there is continuing, growing dissatisfaction among many design engineers with the current policy and general approach to the subject. Such dissatisfaction can be attributed to the problems and costs of meeting current design policy, which have changed in recent years, coupled with a lack of evidence of the safety effectiveness of the policy.

Engineers and researchers have made much progress toward investigating stopping sight distance requirements (*1–3*). Their efforts have been valuable in quantifying important measures of effectiveness and in helping to put SSD in perspective with other design needs. Yet research focus thus far has not addressed the real issue in SSD: the basic model used to determine SSD values for highway design.

This paper is intended to focus the technical debate concerning SSD and to present a new approach to SSD design. This approach represents a major change in policy, yet one that is clearly overdue. In brief, it involves the abandonment of the concept that a single operational model for SSD is appropriate for all highway types under all conditions. Instead, it suggests *functional highway classification* as the foundation for determining SSD design policy and values.

Jack E. Leisch & Associates, 1603 Orrington Avenue, Suite 1200, Evanston, Ill. 60201.

HISTORICAL REVIEW OF SSD

Current basic design policy for SSD has remained unchanged for almost 50 years. It is summarized (*4–6*) as follows:

> The minimum sight distance available on a highway 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. While 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.

AASHTO "Object in Road" Model

The American Association of State Highway and Transportation Officials (AASHTO) model for SSD, formalized in 1940, describes design requirements in simple terms, as shown in Figure 1. The parameters of interest in SSD design include eye height, object height, perception-reaction time, pavement-tire coefficient of friction, and speed of operation. What is notable is that design policy as formulated establishes the same operational model (collision avoidance of an object in the road) and the same values for each parameter, regardless of the type of highway.

Evolution of AASHTO Policy

The AASHTO design policy has changed as the population of vehicles and drivers has changed and as operational and safety research has shed light on safety effectiveness of SSD. Table 1 summarizes the changes in AASHTO policy design

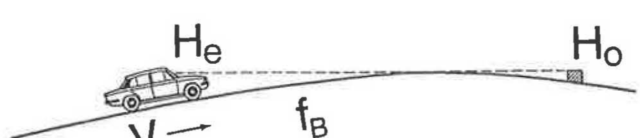

FIGURE 1 AASHTO model for stopping sight distance.

TABLE 1 EVALUATION OF AASHTO STOPPING SIGHT DISTANCE POLICY

	Design Parameters					
Year	Eye Height (ft)	Object Height (in.)	Perception-Reaction Time (sec)	Assumed Tire-Pavement Coefficient of Friction (f)	Assumed Speed for Design	Effective Change from Previous Policy
1940	4.5	4	Variable[a]	Dry—from 0.50 at 30 mph to 0.40 at 70 mph	Design speed	
1954	4.5	4	2.5	Wet—from 0.36 at 30 mph to 0.29 at 70 mph	Lower than design speed (28 mph at 30-mph design speed; 59 mph at 70-mph design speed)	No net change in design distance
1965	3.75	6	2.5	Wet—from 0.36 at 30 mph to 0.27 at 80 mph	Lower than design speed (28 mph at 30-mph design speed; 64 mph at 80-mph design speed)	No net change in design distance
1970	3.75	6	2.5	Wet—from 0.35 at 30 mph to 0.27 at 80 mph	Minimum values—same as 1965; desirable values—design speed	Increase in SSD of up to 250 ft at 70 mph
1984	3.50	6	2.5	Wet—from 0.35 at 30 mph to 0.27 at 80 mph	Minimum values—same as 1965; desirable values—design speed	No net change from 1970

[a]3.0 sec at 30 mph to 2.0 sec at 70 mph.

and parameters and the effects of these changes on actual design.

The most recent changes, reflected in the 1984 policy, have the effect of lengthening the required vertical curve and increasing the required horizontal curve offset for a given design speed. These changes, combined with increasing emphasis on reconstruction problems and costs, have highlighted SSD as a major design concern. In many cases of major reconstruction, an existing alinement must be revised if the design agency desires such reconstruction to be compatible with current policy.

Nevertheless, as pointed out in previous work (*7*), blanket alinement reconstruction is clearly not a cost-effective design approach. Other recent research confirms that, in terms of accidents and safety, there is little reason to believe that SSD design is having the kinds of safety effects that designers believed would occur or intended to occur.

PROPOSED SSD OPERATIONAL MODEL FOR DESIGN OF HIGHWAYS

This paper proposes a revised operational model for stopping sight distance. More precisely, *a series of models* is proposed rather than one single model. This paper outlines the framework for these models and illustrates possible resulting design values for SSD. There are four key elements of the proposed approach:

1. SSD requirements are considered to be related to a number of possible operational events, rather than only one event.
2. There are inherent differences in the operating characteristics and safety experiences of different highway types. The overall design approach should recognize these differences through the use of operating models that relate to each highway type.
3. Human factors and vehicle-roadway parameters also differ with roadway type.
4. SSD 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.

Functional Classification

SSD design models should be based on the functional classification of the highway system. Of primary interest is location (rural or urban), cross section (undivided or divided), general level of traffic volume, and control of access. Solely for the purpose of illustrating the approach here, five distinctly different types of highways are considered—low-volume rural roads, two-lane primary rural highways, multilane urban arterials, rural freeways, and urban freeways.

For each type of highway, there are many critical events that might reasonably serve as the basis for an SSD operational model. Table 2 summarizes these and addresses the concerns of interest—frequency of an occurrence and severity of the consequences of the event. Confronting a large object in the road may be a critical event for design of low-volume, high-speed highways when one considers the lower relative probabilities of vehicle-vehicle conflicts. For other highway types, however, accident and operational experience—as well as common sense—dictate that other more frequent and serious conflicts offer better representations of critical operations. On facilities with uncontrolled access, crossing or rear-end conflicts with stopped vehicles are important. On rural freeways, fewer vehicle-vehicle conflicts occur, making vehicle-object conflicts relatively more important.

For the purpose of discussion, design critical events for the five highway types are presented here. They are

- Low-volume road (LVR)—single-vehicle encounter with a large object (1 ft high);
- Two-lane primary rural highway (2LRP)—vehicle-vehicle conflict involving crossing or stopped vehicle;

TABLE 2 ROADWAY EVENTS RELATED TO SSD

Type of Event	Frequency of Occurrence	Severity of Conflict/ Impact
Two-Lane Rural Highway		
Object in road		
Large animal	Variable—generally infrequent	Severe
Road debris	Infrequent	Minor to moderate
Rocks	Infrequent	Minor
Small animal	Occasional	Minor to moderate
Icepatch	Infrequent	Minor to moderate
Pothole, washout	Infrequent	Minor
Vehicle in road		
Head-on	Very infrequent	Very severe
Rear-end	Frequent	Severe
Crossing	Occasional	Severe
Pedestrian/bicyclist	Very infrequent	Very severe
Rural Freeway		
Object in road		
Large animal	Variable—generally infrequent	Severe
Road debris	Infrequent	Moderate
Rocks	Infrequent	Moderate
Small animal	Infrequent	Moderate
Icepatch	Infrequent	Minor to moderate
Pothole, washout	Infrequent	Minor to moderate
Vehicle in road		
Rear-end	Infrequent	Very severe
Pedestrian/bicyclist	Infrequent	Very severe
Urban Arterial		
Object in Road		
Large animal	Very infrequent	Severe
Road debris	Infrequent	Minor
Rocks	Very infrequent	Minor
Small animal	Infrequent	Minor
Icepatch	Infrequent to occasional	Moderate
Pothole, washout	Occasional	Minor to moderate
Vehicle in road		
Head-on	Infrequent	Very severe
Rear-end	Frequent	Moderate to severe
Crossing	Frequent	Severe
Pedestrian/bicyclist	Frequent	Very severe
Urban Freeway		
Object in road		
Road debris	Frequent	Moderate
Small animal	Very infrequent	Moderate
Icepatch	Infrequent	Moderate to severe
Pothole, washout	Infrequent	Moderate to severe
Vehicle in road		
Rear-end	Frequent	Moderate to severe
Pedestrian	Very infrequent	Very severe

- Multilane urban arterial (MUA)—vehicle-vehicle rear-end conflict;
- Rural freeway (RF)—single-vehicle conflict with small (0- to 6-in.) object; and
- Urban freeway (UF)—vehicle-vehicle conflict (rear-end).

Selection of these design critical events represents an attempt to identify events that would (a) occur frequently enough and (b) result in severe enough consequences that a reasonably cost-effective basis for highway design might ensue.

Driver, Vehicle, and Roadway Characteristics

Operational model parameters require assumptions concerning driver behavior, vehicles, and roadway characteristics. These might also be expected to vary by highway type. Among the parameters of interest are

- Perception-reaction time, $t_{P/R}$;
- Vehicle type(s);
- Assumed deceleration and braking behavior; and
- Available pavement-tire friction.

Perception-Reaction Time

Current design policy assumes 2.5 sec for perception and reaction time and hard braking for collision avoidance with the 6-in. object. Within the framework of functional classification, it is reasonable—in fact desirable—to differentiate in development of model assumptions. Regarding driver behavior, the driver's state of mind has an important effect on performance. Whether the driver is alert or fatigued and what the driver's expectations are for the type of trip and highway should vary by functional classification. A second consideration is the complexity of the driving task, which is strongly related to highway functional type. Uncontrolled-access, high-speed highways present constant decisions to drivers. Rural freeways by their very nature are easy to drive. Urban freeways, because of the density of traffic and frequency of interchanges, are relatively more difficult to drive.

The AASHTO policy discusses driver reaction time in the context of whether information is expected or unexpected and the distribution of driver reaction behavior. The median driver reaction time for responding to unexpected, simple information is about 1.5 sec. More complex decisionmaking and consideration of 85th-percentile versus median drivers result in reaction times of 5 sec or more.

There are clearly differences in the types of decisions, the state of drivers, and the need or desire to design for 85th- versus 50th-percentile behavior. The example below illustrates how a different set of driver assumptions could translate to a range of assumed perception-reaction times for the various highway types studied here.

	LVR	*Two-Lane Primary (Rural)*	*Urban Arterial*	*Rural Freeway*	*Urban Freeway*
Driver's state	Alert	Fatigued	Alert	Fatigued	Fatigued
Complexity of tasks	Low	Moderate	High	Low	High
Assumed $t_{P/R}$ (sec)	1.5	3.0	2.5	2.5	3.0
Percent $t_{P/R}$ of AASHTO	60	120	100	100	120

Vehicle Type

AASHTO SSD requirements are based solely on passenger car characteristics. Some recent research has challenged the AASHTO assumptions that the added height of a truck driver's eye compensates for his vehicle's longer stopping requirements. Furthermore, in terms of design for horizontal SSD requirements, a truck driver's greater eye height offers no advantage. There is evidence, however, that advancing brake technology will soon produce truck stopping distances that are much shorter than those produced by the current fleet.

A revised set of models for SSD policy should fully investigate the truck-passenger car sensitivities. SSD design for rural primary highways and urban highways may need to be based on truck rather than passenger car characteristics.

Deceleration and Braking

There are two aspects to an assumed design value for the friction factor. The first is the friction capability of the pavement. To the extent that this tends to vary for different highway types, the operational model's friction chracteristics should vary. The second aspect is the assumed or desired driver action. Is a hard braking response to every event on all highway types a reasonable or desirable model assumption? The model proposed here again differentiates among highway classes. Higher-class facilities could be designed assuming a greater standard of comfort than is assumed for lower classes. The lowest-class facilities, in turn, should probably be designed with minimal consideration for driver comfort. This is consistent with the approach intended by AASHTO in presenting design policy by functional classification.

When a range in assumed friction characteristics and differences in assumed driver braking behavior are used in model development, further SSD variation can be expected. For discussion purposes, the design values for f shown in Figure 2 were used to compute SSD. These values are consistent with the rationale discussed above—that lower-class facilities should be designed with minimal consideration for driver comfort. Note that to simplify the presentation, only passenger car braking is assumed. Truck behavior, as stated previously, may be a better basis for SSD design for some highway types. Analysis of the possible effects of design for trucks was beyond the scope of this paper.

Stopping Sight Distance Requirements

Once design models and parameters have been selected, it is possible to calculate SSD requirements by functional highway

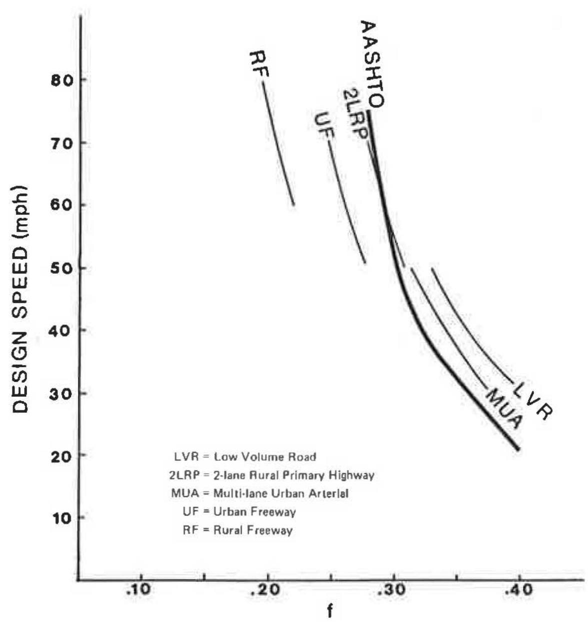

FIGURE 2 Design values for coefficient of friction by functional class.

TABLE 3 STOPPING SIGHT DISTANCE REQUIREMENTS BY HIGHWAY TYPE

Design Speed (mph)	Low-Volume Road[a]			Two-Lane Primary Rural Highway[b]			Multilane Urban Arterial[c]			Urban Freeway[b]			Rural Freeway[c]		
	$d_{P/R}$	d_B	SSD	$d_{P/R}$	d_B	SSD	$d_{P/R}$	d_B	SSD	$d_{P/R}$	d_B	SSD	$d_{P/R}$	d_B	SSD
30	66	75	141				110	79	189						
40	88	148	236	176	167	343	147	157	304						
50	110	253	363	220	278	498	183	269	452	220	298	518	183	362	545
60	132	375	507	264	414	680				264	462	726	220	545	765
70				308	583	891				308	681	989	257	817	1,074

NOTE: $d_{P/R}$, d_B, and *SSD* values in feet.
[a] $t_{P/R}$ = 1.5 sec.
[b] $t_{P/R}$ = 3.0 sec.
[c] $t_{P/R}$ = 2.5 sec.

type and design speed. The standard equations shown in the AASHTO policy (*6*) are used:

$$SSD = d_{P/R} + d_B$$

$d_{P/R}$ = distance traveled during perception-reaction time (ft)
d_B = distance traveled while braking (ft)
$d_{P/R} = 1.47\ V_{\text{Des}} t_{P/R}$
$d_B = (V_{\text{Des}})^2/30f$
V_{Des} = design speed (mph)
$t_{P/R}$ = perception-reaction time (sec) (above)
f = design coefficient of friction for braking (Figure 2)

If SSD design values are calculated for each class of highway using the above assumptions, the values shown in Table 3 result. Note that the cumulative effect of varying the parameters results in a range of stopping sight distances from 363 to 545 ft for 50-mph highways. This produces values that are from 79 to 118 percent of current AASHTO policy (see Figure 3).

It is also important to note the relationship between functional class and stopping sight distance. Much lower values than AASHTO recommends are shown for low-volume rural roads. Somewhat *higher* values than AASHTO recommends are called for in the case of two-lane primary rural highways. The greatest values are indicated for rural freeways. Although one might argue with the relative spread or specific parameter values used, the overall results appear logical, and they are consistent with many engineers' views of design. The values illustrate what is considered to be the desired result, that is, a meaningful variation in SSD by functional class.

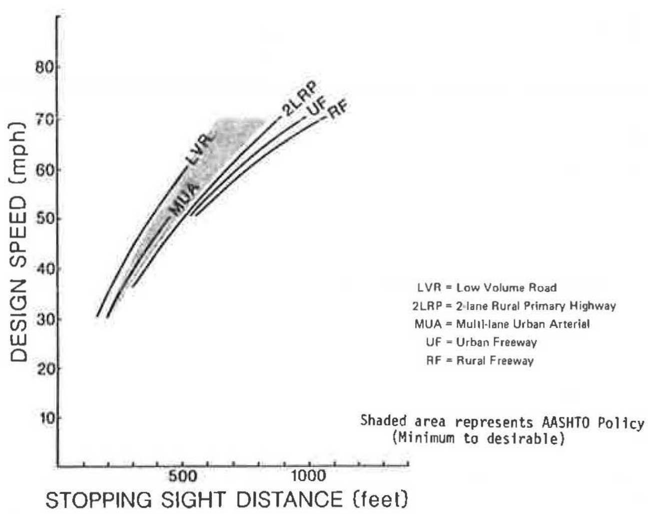

FIGURE 3 Stopping sight distance design values by highway type.

TRANSLATION OF SSD VALUES TO HIGHWAY DESIGN REQUIREMENTS

The final step in the new approach to SSD design is development of design lengths for vertical curves and offsets through horizontal curves. Here again, variations in the functional SSD models may produce variable design results. Such variation reflects the lack of one single object or eye height assumption that is appropriate for all highway types.

Vertical Curve Design

The functional operational models previously presented imply the eye and object heights shown below. (The author has not evaluated SSD requirements for trucks for this paper. Greater eye heights would be used, along with much longer SSD values. This sensitivity should be investigated, and selection of a rational model made.) Design values of 2.0 ft for tail lights and 1.0 ft for a large object are used.

Vertical curve length requirements are calculated using the following (*6*):

For SSD < L

$$L = A\,(\text{SSD})^2/100\,[(2h_1)^{1/2} + (2h_2)^{1/2}]^2$$

For SSD > L

$$L = 2(\text{SSD}) - 200\,[(h_1)^{1/2} + (h_2)^{1/2}]^2/A$$

where

L = length of crest vertical curve (ft),
A = algebraic difference in grades,
h_1 = height of eye (ft), and
h_2 = height of object (ft).

Tables 4–8 show vertical curve length design values for the full range of design speeds. To illustrate the variability in design, consider Figure 4, which shows a plot of crest vertical

TABLE 4 DESIGN LENGTH REQUIREMENTS FOR CREST VERTICAL CURVES ON LOW-VOLUME ROADS—LENGTH OF VERTICAL CURVE IN FEET

Algebraic Difference in Grades %	Design Speed (mph) 30 (SSD=141 ft.)	40 (SSD=236 ft.)	50 (SSD=353 ft.)	60 (SSD=507 ft.)	70 (SSD= -- ft.)
1	90	120	150	180	-
2	90	120	150	190	-
3	90	120	177	465	-
4	90	120	314	624	-
5	90	142	400	760	-
6	90	197	480	936	-
7	90	237	560	1092	-
8	90	270	640	1248	-
9	99	304	719	1404	-
10	117	338	799	1553	-

Design assumptions:

Coefficient of braking friction per Figure 2
Perception/reaction time = 3.0 sec.
Height of Object = 2.0 ft.
Height of Eye = 3.5 ft.

Numbers above the line represent minimum curve lengths based on Length = 3 x V_{Des}

TABLE 5 DESIGN LENGTH REQUIREMENTS FOR CREST VERTICAL CURVES ON TWO-LANE RURAL PRIMARY HIGHWAYS—LENGTH OF VERTICAL CURVE IN FEET

Algebraic Difference in Grades %	Design Speed (mph) 30 (SSD= -- ft.)	40 (SSD=343 ft.)	50 (SSD=498 ft.)	60 (SSD=680 ft.)	70 (SSD=891 ft.)
1	-	120	150	180	210
2	-	120	150	281	703
3	-	120	277	641	1103
4	-	146	456	857	1471
5	-	254	575	1071	1639
6	-	326	629	1285	2207
7	-	362	804	1500	2575
8	-	436	919	1714	2943
9	-	491	1034	1928	3310
10	-	545	1149	2142	3678

Design assumptions:

Coefficient of braking friction per Figure 2
Perception/reaction time = 3.0 sec.
Height of Object = 2.0 ft.
Height of Eye = 3.5 ft.

Numbers above the line represent minimum curve lengths based on Length = 3 x V_{Des}

curve length design values for the five functional models compared with current AASHTO policy for 50-mph design speed. What is interesting is the great variation in length requirements. For example, for a low-volume road with a A of 6 and 50-mph design speed, the vertical curve length requirement is 480 ft, compared with the AASHTO values of 660 ft to 960 ft. Rural freeway vertical curve requirements would be much greater under the model assumptions—1,340 ft. This results from the use of a 6-in. object height for rural freeways rather than a 1.0- or 2.0-ft object height.

TABLE 6 DESIGN LENGTH REQUIREMENTS FOR CREST VERTICAL CURVES ON MULTILANE URBAN ARTERIALS—LENGTH OF VERTICAL CURVE IN FEET

Algebraic Difference in Grades %	Design Speed (mph) 30 (SSD=189 ft.)	40 (SSD=304 ft.)	50 (SSD=452 ft.)	60 (SSD= -- ft.)	70 (SSD= -- ft.)
1	90	120	150	-	-
2	90	120	150	-	-
3	90	120	185	-	-
4	90	120	364	-	-
5	90	176	473	-	-
6	90	248	568	-	-
7	90	300	663	-	-
8	108	343	757	-	-
9	138	385	852	-	-
10	162	428	947	-	-

Design assumptions:

Coefficient of braking friction per Figure 2
Perception/reaction time = 2.5 sec.
Height of Object = 2.0 ft.
Height of Eye = 3.5 ft.

Numbers above the line represent minimum curve lengths based on Length = 3 x V_{Des}

TABLE 7 DESIGN LENGTH REQUIREMENTS FOR CREST VERTICAL CURVES ON URBAN FREEWAYS—LENGTH OF VERTICAL CURVE IN FEET

Algebraic Difference in Grades %	Design Speed (mph) 30 (SSD= -- ft.)	40 (SSD= -- ft.)	50 (SSD=518 ft.)	60 (SSD=725 ft.)	70 (SSD=989 ft.)
1	-	-	150	180	210
2	-	-	150	373	899
3	-	-	317	733	1360
4	-	-	496	977	1813
5	-	-	622	1221	2266
6	-	-	746	1465	2719
7	-	-	870	1709	3172
8	-	-	995	1954	3626
9	-	-	1119	2198	4079
10	-	-	1243	2442	4532

Design assumptions:

Coefficient of braking friction per Figure 2
Perception/reaction time = 3.0 sec.
Height of Object = 2.0 ft.
Height of Eye = 3.5 ft.

Numbers above the line represent minimum curve lengths based on Length = 3 x V_{Des}

Horizontal Offsets

The minimum offset from the outside lane to a roadside obstruction to provide horizontal SSD is given by the following from the AASHTO policy (*6*):

$$M = (5730/D)\ [1 - \cos\ (\text{SSD} \times D/200)]$$

where M = offset from center of lane to obstruction (feet) and D = degree of horizontal curve. Design values here are solely a function of SSD values (Table 3) and not eye and

TABLE 8 DESIGN LENGTH REQUIREMENTS FOR CREST VERTICAL CURVES ON RURAL FREEWAYS—LENGTH OF VERTICAL CURVE IN FEET

Algebraic Difference in Grades %	Design Speed (mph) 30 (SSD= -- ft.)	40 (SSD= -- ft.)	50 (SSD=545 ft.)	60 (SSD=765 ft.)	70 (SSD=1074 ft.)
1	-	-	150	201	819
2	-	-	425	881	1736
3	-	-	670	1321	2603
4	-	-	894	1761	3471
5	-	-	1117	2202	4339
6	-	-	1341	2642	5207
7	-	-	1564	3082	6075
8	-	-	1788	3522	6943
9	-	-	2011	3963	7810
10	-	-	2235	4403	8678

Design assumptions:

Coefficient of braking friction per Figure 2
Perception/reaction time = 2.5 sec.
Height of Object = 0.5 ft.
Height of Eye = 3.5 ft.

Numbers above the line represent minimum curve lengths based on Length = 3 x V_{Des}

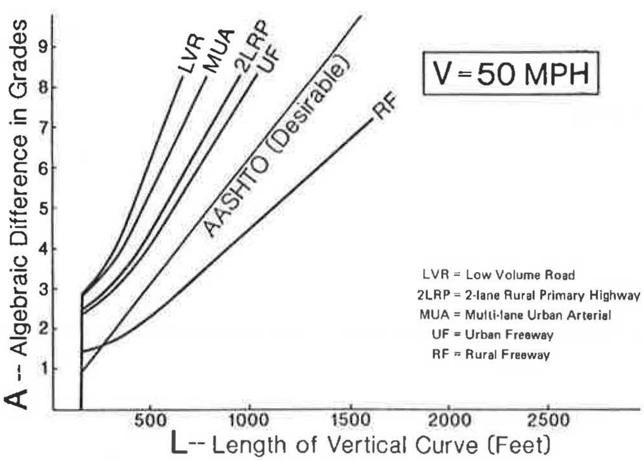

FIGURE 4 Comparison of crest vertical curve design requirements for 50-mph design speed.

object height parameters. The resulting values for the range of design speeds are shown in Table 9.

REFINEMENTS AND ADJUSTMENTS TO REFLECT SPECIAL GEOMETRY OR CONDITIONS

Current design policy for SSD does not account for any of the operational variations in safe stopping requirements that actually occur along a highway. The present policy produces designs that are inconsistent operationally and inevitably not cost effective. The inclusion of a range of values to reflect operational variations is suggested as an important element of the recommended new approach to SSD.

There are two aspects to be considered. The first is the effect of confounding geometry or unusual conditions within the influence of the area of limited sight distance. Examples include the presence of intersections, diverges, horizontal curvature, changes in cross section, and the like. At these locations, additional sight distance should be routinely provided. Alternative design values to the base values presented earlier should be derived on the basis of rationally derived alternative values for the particular operational parameters of different highway types. To illustrate, consider the following possible adjustments (NC indicates no change over values recommended previously):

Highway Type	*Condition with SSD Constraint*	*Adjustments to Operational Model Parameters* $t_{P/R}$ *(sec)*	h_1 *(ft)*	h_2 *(ft)*	f
Low-volume road	Intersection, sharp horizontal curve	4.0	3.5	0	NC
Two-lane primary	Major intersection, sharp curve	5.0	3.5	0	NC
Urban arterial	Change in cross section, major intersection, sharp horizontal curve	6.0	3.5	0	NC
Rural freeway	Interchange	7.0	3.5	0	NC
Urban freeway	System or major interchange	7.0	3.5	0	NC

TABLE 9 HORIZONTAL CURVE OFFSETS (M) REQUIRED FOR SSD

Design Speed (mph)	AASHTO Model Min. SSD[a]	M	Des. SSD[b]	M	Low-Volume Road SSD	M	Two-Lane Primary Rural Highway SSD	M	Multilane Urban Arterial SSD	M	Urban Freeway SSD	M	Rural Freeway SSD	M
30	—	—	200	21.3	141	10.7			189	19.1				
40	275	21.5	325	30.0	236	15.9	343	33.3	304	26.2				
50	400	25.0	475	39.7	363	23.3	498	43.6	452	36.0	518	47.2	545	52.1
60	525	30.8	650	47.1	507	28.8	680	51.5			726	58.7	765	65.1
70	625	31.5	850	58.0			891	63.7			989	78.4	1,074	92.3

NOTE: SSD and M values are in feet.
[a]Min. SSD = minimum SSD based on assumed running speeds for wet pavement lower than full design speed (6).
[b]Des. SSD = desirable SSD based on assumed full design speed (6).

TABLE 10 STOPPING SIGHT DISTANCE REQUIREMENTS FOR LOCATIONS WITH SPECIAL GEOMETRY OR CONDITIONS

Design Speed (mph)	Low-Volume Road[a]			Two-Lane Primary Rural Highway			Multilane Urban Arterial			Urban Freeway			Rural Freeway		
	$d_{P/R}$	d_B	*SSD*	$d_{P/R}$	d_B	*SSD*	$d_{P/R}$	d_B	*SSD*	$d_{P/R}$	d_B	*SSD*	$d_{P/R}$	d_B	*SSD*
30	176	75	251				265	79	344						
40	235	148	383	294	167	461	353	157	510						
50	294	253	547	368	278	646	441	269	710	514	298	812	514	362	876
60	352	375	727	441	414	855				617	462	1,079	617	545	1,162
70				514	583	1,097				720	681	1,401	720	817	1,537

NOTE: $d_{P/R}$, d_B, and *SSD* values in feet.

The longer perception-reaction times are consistent with the unexpected and more complex driver decisions produced by the special condition. Similarly, an object height of 0 ft represents a rational requirement to see the pavement or geometry that contributes to the special condition. When these adjustments are used in the calculation of stopping sight distance requirements, the results shown in Table 10 would apply to design.

The implications of these adjustments are clear. In certain locations, regardless of the type of highway, stopping sight distance requirements are greater. This is because of special circumstances that may require additional time for drivers to make decisions or react. Why not formulate design policy to explicitly recognize these additional needs? On the other hand, it is undoubtedly costly, difficult, and, in the long run, counterproductive to formulate SSD design policy around a single most critical model. In most locations the long SSD values produced by the above parameters would clearly not be justified by the costs of achieving such values.

Other adjustments should also be made to reflect the dynamic requirements of braking on a curve or stopping on a downgrade. These adjustments would apply whenever the segment of restricted stopping sight distance coincides with moderate to severe horizontal or vertical alinement. Here, revised design values for the coefficient of braking friction can rationally produce adjusted design values.

RECONSTRUCTION VERSUS NEW CONSTRUCTION

The SSD issue cannot be completely addressed without mention of problems associated with reconstruction. The current AASHTO policy specifies that "this publication is intended to provide guidance in the design of new and major reconstruction projects." The design profession is thus faced with a dilemma that seriously affects design, budgeting, and programming functions. Given the changes in design policy previously described, every time a major reconstruction project occurs, one of three difficult choices must be made:

1. Redesign the alinement to upgrade it to current SSD policy;
2. Ignore any deficiencies in SSD (as measured against current policy) and reconstruct on existing alinement; or
3. Evaluate each segment of alinement and either reconstruct or request a "design exception."

The first approach is extremely costly. The second inevitably produces problems with tort liability. The third, undoubtedly the best approach, is time-consuming. Moreover, when engineers evaluate existing SSD-deficient locations, most often there is no safety problem identified. A rational decision based on such analysis is to request a design exception. Design exceptions have unfortunately become routine in 4R projects, rather than special or unusual cases. This is not the fault of location and design engineers, but rather the inevitable result of a flawed design policy.

The solution to this dilemma is to treat new construction SSD design differently from reconstruction within the framework of the policy. A rational decision, backed up by analysis of site conditions and actual safety, should not have to be labeled as a design exception. Instead, design values and procedures should be determined in a manner that is sensitive to the particular difficulties and aspects of major reconstruction.

SUMMARY

This paper was intended to provide the design profession with a fresh approach to stopping sight distance. Example parameter and design volumes were presented to illustrate the model concepts and to demonstrate the sensitivities that should be a part of stopping sight distance design policy. At this stage, the exact values cannot be fixed, but should be extensively tested through further research. Rather than focus on these values, the author urges researchers and designers to address the following concepts:

1. The existing AASHTO operational model for stopping sight distance is not reflective of reasonably frequent occurrences of critical events for all highway types.
2. There are inherent differences in sight distance requirements among highway types defined by their location, traffic volume, cross section, and access control. Such differences should be part of any operational model or models for SSD.
3. Differences among highway types are also reflected in differences in assumed driver behavior and dynamic vehicle characteristics. Basic design parameters should vary for the range of highway types.
4. Design for horizontal and vertical SSD should reflect additional operational needs imposed by confounding geometry.
5. SSD design values should be separately derived for major reconstruction versus new construction.

The most recent edition of the AASHTO policy provides an ideal framework for presenting a functionally classification-based SSD design policy. Concepts related to operational models, driver-vehicle design, parameters, and other basics can be presented in Chapter II. Each individual chapter could then contain separately derived design tables and charts for SSD.

Should the approach presented here be adopted for highway design, there would be much greater flexibility within the presentation of standard values. Much more cost-effective designs would result, providing additional sight distance where it is most needed. Such cost-effectiveness would be achieved *within* the framework and values rather than through design exceptions in the policy.

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Safety Effects of Limited Sight Distance on Crest Vertical Curves

THOMAS URBANIK II, WANDA HINSHAW, AND DANIEL B. FAMBRO

The safety effects of limited sight distance at crest vertical curves on two-lane rural highways in Texas were examined. Two large data bases consisting of 222 study segments of approximately one-mile lengths representing nearly 1,500 accidents were assembled to evaluate the effects that stopping sight distance along crest vertical curves has on accident rates. It was found that the relationship between available sight distance on crest vertical curves on two-lane roadways and accidents is difficult to quantify, even when a large data base exists. The AASHTO stopping sight distance design model alone is not a good indicator of accident rates on two-lane roadways in Texas; thus, use of this model alone will not result in cost-effective project design. Where there are intersections within the limited sight distance sections of crest vertical curves, there is a statistically significant increase in accident rates. It can be inferred that other geometric conditions within limited sight distance sections of crest vertical curves could also cause a marked increase in accident rates. An example would be a sharp horizontal curve hidden by a crest vertical curve. The increase in accident rates because of intersections within limited sight distance sections of crest vertical curves was more pronounced on roadways with higher volumes, implying that a threshold volume level may be determined based on considerations of cost effectiveness.

This paper reports the results of an accident analysis on two-lane, two-way rural roads in Texas with 55 mph speed limits to determine the effect of crest vertical curve lengths on the number of accidents. The minimum design control for crest vertical curve length is the stopping sight distance. Stopping sight distance is calculated using basic principles of physics and the relationships among various design parameters. AASHTO defines stopping sight distance as the sum of two components: brake reaction distance (distance traveled from the instant of object detection to the instant the brakes are applied) and the braking distance (distance required for the vehicle to come to a complete stop). Crest vertical curves with stopping sight distance less than 450 ft are considered to have limited sight distance at 55 mph based on current AASHTO policy (*1*). In order to assess the cost effectiveness of reconstruction projects to upgrade vertical alignment, it is necessary to know the safety impacts of limited sight distance on crest vertical curves.

Texas Transportation Institute, Texas A&M University, College Station, Tex. 77834.

PREVIOUS RESEARCH ON SAFETY AND STOPPING SIGHT DISTANCE

The vertical alignment of a highway is a balance of cost and safety. The effects of grade and stopping sight distance on accident rates on vertical curves have been analyzed in a number of studies. The results of the previous studies (*2–16*) appear to be inconclusive and inconsistent.

The difficulty of obtaining adequate data to evaluate the effects of limited sight distance on accident occurrence is surely a significant cause of the inconsistency of previous research. Several factors contribute to the difficulty. The extreme variability seen in accident rates, even under carefully controlled circumstances, makes the detection of any effect of limited sight distance extremely difficult. In addition, the availability of sites necessary to the design of meaningful comparison studies is limited because of the need to control for all elements at or near the stopping sight distance restriction.

If adequate controls are not used, the accident data recorded may reflect other geometric elements, such as intersections. This result is partially the result of the difficulty of defining adequately homogeneous sites. Additionally, it may be caused by accident data that are not recorded with enough precision to allow association between particular accidents and the short lengths of roadway that exhibit sight distance restrictions. This limitation sometimes necessitates the use of an overall segment accident rate instead of the rate associated exclusively with the short distance exhibiting the sight restriction as the measure of the effect of limited stopping sight distance. Because there may be relatively few sight restrictions relative to the length of roadway, an overall segment accident rate may dilute any effect of the stopping sight distance restrictions within the segment. And, as seen in the present study, the effects of sight distance restrictions may only be seen through their interaction with other geometric features, again making their detection more difficult. All these factors help to explain the inconsistencies seen in this review of the literature.

STUDY DESIGN

The initial study design was based on identifying the largest possible data base consisting of rural two-lane highways with and without limited sight distance. Potential study areas were identified in east and central Texas, where sufficient topographic relief was known to occur and limited sight distance segments were believed to exist.

Criteria were established for selecting potential study segments. The segment criteria included posted speed limit, proximity to signalized intersections, and segment length. The posted speed, including horizontal curves, had to be 55 mph or greater. The study segment could not be within one-half mile of a signalized intersection. The minimum segment length was set at one mile to eliminate short segments that might be overly affected by adjacent segments. These criteria were believed to be reasonable for controlling a number of factors (for example, horizontal curves and intersections) that could mask the effects of crest vertical curves. Specifically, horizontal curves and intersections are known contributing factors to accidents.

METHODOLOGY

In order to investigate the potential relationship between accident rate and limited sight distance caused by crest vertical curves, sections of highway with varying amounts of limited sight distance were identified and grouped by road type. Two general road types—two-lane with shoulders and two-lane without shoulders—produced sufficient lengths of roadway for analysis.

The initial selection of highway sections was restricted in an attempt to produce groups with segments as homogeneous as possible. The selection included only rural highways, and the geometry of each was carefully inspected to ensure conformity to predetermined standards as previously stated. Every segment was visited and videotaped.

Highway profiles were used to identify all vertical curves on the selected roadways and to characterize them by their length and K factor. Horizontal curves were also identified and the length and degree of curvature of each were recorded. Segments of approximately one mile in length were then defined on the sample roadways. These segments were used throughout the analysis as the experimental observations. The original roadway lengths were divided into these segments with the limitation that no vertical curve or horizontal curve was broken into two segments. Thus, the segment lengths were kept as close as possible to one mile while honoring this restriction.

The intersecting roads on each segment were counted and classified as numbered roads, county roads, or driveways, based on information available from the highway profiles. It was noted from actual observation of the sites that not all driveways were included on the plans.

The relative amounts of limited sight distance were calculated from the recorded data on crest vertical curves for all segments. Three criteria were used to define limited sight distance. AASHTO policy specifies a minimum stopping sight distance for a wide range of design speeds. The minimum values for 45 mph (325 ft), 55 mph (450 ft), and 65 mph (550 ft) were used to calculate limited sight distance. The AASHTO policy also specifies a desirable stopping sight distance. The desirable value for 55 mph, which is the posted speed on all roadways in this study, is 550 ft—or the minimum value for 65 mph.

The length of roadway that was calculated to be limited for each segment according to each criterion was translated into the percent of the road segment that was judged limited according to the various distance criteria. These measures of the relative amount of sight distance in the road segments were used to evaluate the effects of crest vertical curves on accident rates. In addition, the state numbered roads, county roads, and driveways on the segments were categorized according to whether they were within the limited sight distance portions of crest vertical curves based on the three criteria for stopping sight distance.

Texas accident data files, collected through the Texas Department of Public Safety and maintained by the Accident Analysis Division of the Texas Transportation Institute, provided the accident history for the selected highway sections. All accidents, with the exception of those reported by the driver, were considered in the calculation of accident rates.

Four years of accident data, 1984 through 1987, were summarized for the analysis. Several years of data were desirable because of the extreme variability in accident rates, even in a carefully selected, homogeneous sample. The rates become more stable over several years and the incidence of zero rates is practically eliminated, which simplifies the analysis. To avoid the possibility of changes in the condition of the selected roadways a longer time interval was not used. It was verified that no construction occurred on the selected sites during the four years of the study.

In addition, the computerized state roadway inventory files were used as the source of traffic volume for the analysis. If the annual average daily traffic (AADT) varied within the defined road segment, an average value was calculated. An average for the segment for the years considered in the analysis was then computed for use in adjusting accident rates for AADT. A match with the roadway inventory (RI) files also ensured the valid identification of the roadway segments using the method of milepoints within control sections.

The approximate one-mile segments served as the sampling unit for the analysis. Data from the three sources—highway profiles, accident data files, and roadway inventory files—were summarized by road segment and merged to produce the complete data set for analysis.

Other units of measurement were considered and rejected because of the inherent limitations of the data. If one could identify accident locations exactly on the roadway, their relative positions with respect to crest vertical curves could be known. This knowledge would allow a direct comparison between accident rates on crest vertical curves and on sections of roadway with flat vertical alignment. This method of comparison was rejected because the recorded accident locations were not believed to be precise. The somewhat arbitrary one-mile segment length was selected to generate as large a sample as possible without going beyond the known limitations of the data.

STATISTICAL METHODS

Multiple regression techniques were employed to investigate and measure the effects of limited sight distance on accident rates. Two types of accident rates were considered as dependent variables in the analysis: accidents per mile and accidents per million vehicle miles (MVM). In both cases, it is of prime importance to adequately model the effect of AADT on the rate before attempting to evaluate other potential effects. Without first adjusting for AADT, examination of the possible effects of limited sight distance is not meaningful. Multiple regression provides the methodology for making these simultaneous adjustments and the associated tests.

Certain assumptions must be met before the use of a least-squares regression analysis is valid. The first is that the observations on the dependent variable are independent. There is no reason to believe that the observations of accident rates for the different road segments in this analysis are not independent. Nevertheless, one could not use multiple observations from consecutive years and comply with this assumption. Thus, this requirement provides another reason to summarize the several years of accident data for each segment into a single observation.

Another assumption that must be met is that the dependent variable, in this case accident rate, is normally distributed with constant variance. The least-squares analysis is robust against deviations in the normality requirement; that is, if the assumption is not strictly met, the analysis is still valid. But, if the assumption of equal variance is not met, the analysis may be flawed.

Accident rates are generally believed to follow a Poisson distribution, not a normal distribution. Additionally, it is known that the Poisson distribution has a variance that is equal to its mean. In other words, as the accident rate increases, the variance increases. Therefore the assumption of equal variance is also violated.

Averaging the numbers of accidents over several years makes the distribution more nearly normal, and taking the logarithm of the rates prior to analysis helps to eliminate the problem of unequal variance or heteroscedasticity. Therefore, instead of accidents per mile a year and accidents per million vehicle miles, the analysis uses the logarithms of both these variables. In order to accommodate the few zero accident rates, the logarithm of the accident rate plus one was used. The adjustments are believed to make the analysis statistically valid.

A nominal significance level of 0.05 is used in interpreting the statistical analyses. This value means that there is only a 5 percent chance of making an error in stating that a given relationship is nonzero. The actual significance probabilities are reported in many cases to allow the reader further interpretation of the results. Also, due to the limited data available for some tests, results that approach significance ($0.05 < p < 0.10$) will be noted.

RESULTS

Two-Lane Roads with Shoulders

One hundred and sixty-eight road segments were defined from the group of two-lane roads with shoulders. Nine hundred and ninety accidents had occurred on these combined segments, the average annual accident rate per mile varying between 0.0 and 8.25. Averaged AADT values ranged between 943 and 9,075. Table 1 gives the frequency of road segments within specified AADT intervals.

Figure 1 provides a plot of accident rate per mile versus AADT. Several observations can be made from this graph. First, the strong relationship between accident rate and AADT is illustrated. Second, the increasing variance as the average accident rate increases can be seen. Last, the tremendous variation in accident rates for fixed AADT can be noted. It is the explanation of this variability that is attempted through the additional variables in the multiple regression analysis, including the measurements of limited sight distance.

TABLE 1 TWO-LANE ROADWAYS WITH SHOULDERS—FREQUENCY AND PERCENT OF SEGMENTS BY AADT

AADT	Freq	Percent
≤ 2000	15	8.9
2-2999	36	21.4
3-3999	45	26.8
4-4999	40	23.8
5-5999	21	12.5
≥ 6000	11	6.5
Total	168	100.0

The relative amounts of limited sight distance varied greatly depending on the criteria used to define adequate sight distance. The percentages of limited sight distance are summarized for the three criteria examined in Table 2. Only two road segments contained lengths with limited sight distance using the lowest criterion—325 ft stopping sight distance. Thus, no analyses could be performed based on sight distance of less than 325 ft because of the lack of data. That is to say, virtually all two-lane roadway segments with shoulders meet the AASHTO minimum criteria for 45 mph. The other two sight distance criteria yielded adequate numbers of segments for analysis, although the majority are not sight deficient by any of the three standards.

The effects of limited sight distance using the criterion of 450 ft, the minimum value for 55 mph, will be examined first. The terminology "percent limited stopping sight distance" will hereafter be used to indicate the percent of the roadway that has less than the specified stopping sight distance based on the current AASHTO driver eye height (3.5 ft) and object height (0.5 ft). Figure 2 examines the possible relationship between accident rate per mile and this measurement of limited sight distance. The percent of roadway with limited sight distance ranges as high as 35 percent, but very few segments have more than 20 percent limited sight distance. No strong relationship can be seen between the average accident rate and percent limited stopping sight distance. From Table 2 it can be seen that 134—or 80 percent—of the road segments have no limitation of sight distance according to this standard.

The relationship between percent limited stopping sight distance and AADT is illustrated in Figure 3. There is no association apparent in this figure. In other words, the sample data set is well balanced with respect to these two variables. The presence of limited stopping sight distance is not associated with only particular values of AADT, but is well represented across the full range—between 2,000 and 8,000 vehicles daily. This balance contributes to confidence in the analytical results.

Regression analyses were performed on the logarithms of accident rate per mile and accident rate per MVM. Included among the independent variables examined were AADT, the square of AADT, percent limited stopping sight distance,

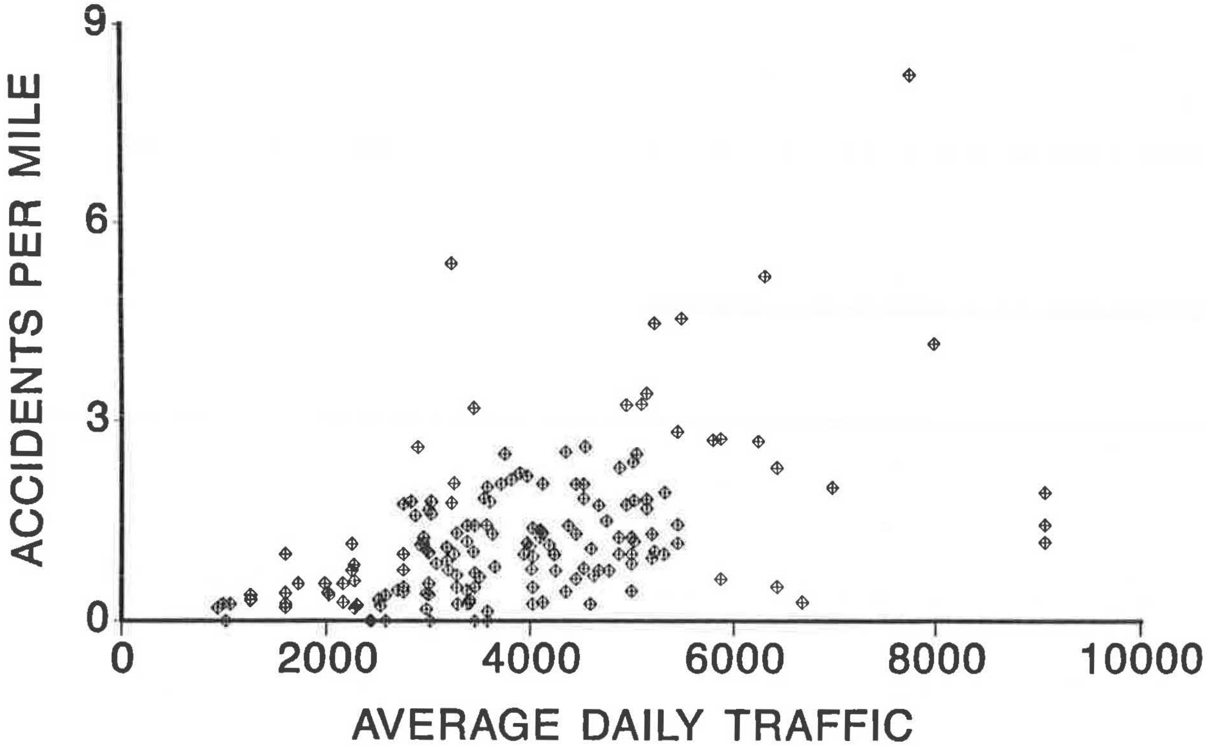

FIGURE 1 Accident rate per mile versus AADT, two-lane roads with shoulders.

TABLE 2 TWO-LANE ROADWAYS WITH SHOULDERS—FREQUENCY OF SEGMENTS BY THREE CRITERIA OF SIGHT DISTANCE

Percent Limited Distance	Minimum 325	Sight 450	Distance (ft) 550
0	166	134	101
1-10	1	16	13
11-20	1	12	32
21-30	0	4	12
31-40	0	2	7
41 +	0	0	3
Total	168	168	168

classification variables identifying the type of intersecting roads on the segment, and the number of intersecting roads within the limited sight distance sections of crest vertical curves. Interactions among these variables were also considered as potential contributors to the models.

Linear terms in the regression model become multiplicative factors when the results are transformed back to the original scale of the data. This results from the original logarithmic transformation of accident rates. Examples of predictive values are provided to aid in interpreting the results. Potential predictive factors are modeled as either continuous variables, such as AADT and percent limited distance, or as categorical variables, such as the types of intersecting roads on a segment.

The classification of segments according to the types of major intersections divided the road segments into four groups. Major intersections were initially placed in one of two categories designated numbered or county roads—and the cross-classification of these two types produced the four possible groups. For example, one group represents segments that contain a county road, but not a numbered road; another group, segments that contain both numbered and county roads. It can be seen throughout the results that this categorical factor contributes to explaining variability in the accident rates before considering the factors of major interest in this study.

The number of intersecting roads that are within the limited stopping sight distance sections of crest vertical curves is considered as a separate continuous variable. All intersections, including the less prominent ones designated as driveways, are counted in this calculation. Only a small percentage of total intersections satisfy the condition of being within a limited stopping sight distance section. In the two-lane-with-shoulder data set, only 19 of 299 roads—or 6 percent of the total intersections—are within sight-deficient curves using the sight distance criterion of 450 ft. Table 3 gives the full summary of available data on intersecting roads.

The results of the analysis of the logarithm of accidents per mile are presented in detail. The accident rate significantly depended on AADT, modeled by a quadratic relationship. The type of intersecting roads also contributed to explaining the variability in accident rates. Using the AASHTO minimum criterion for 55 mph of 450 ft, the variable for stopping sight distance, percent limited stopping sight distance, was not significantly associated with accidents per mile after adjustment for these two factors. The variable for the number of intersecting roads within limited stopping sight sections of

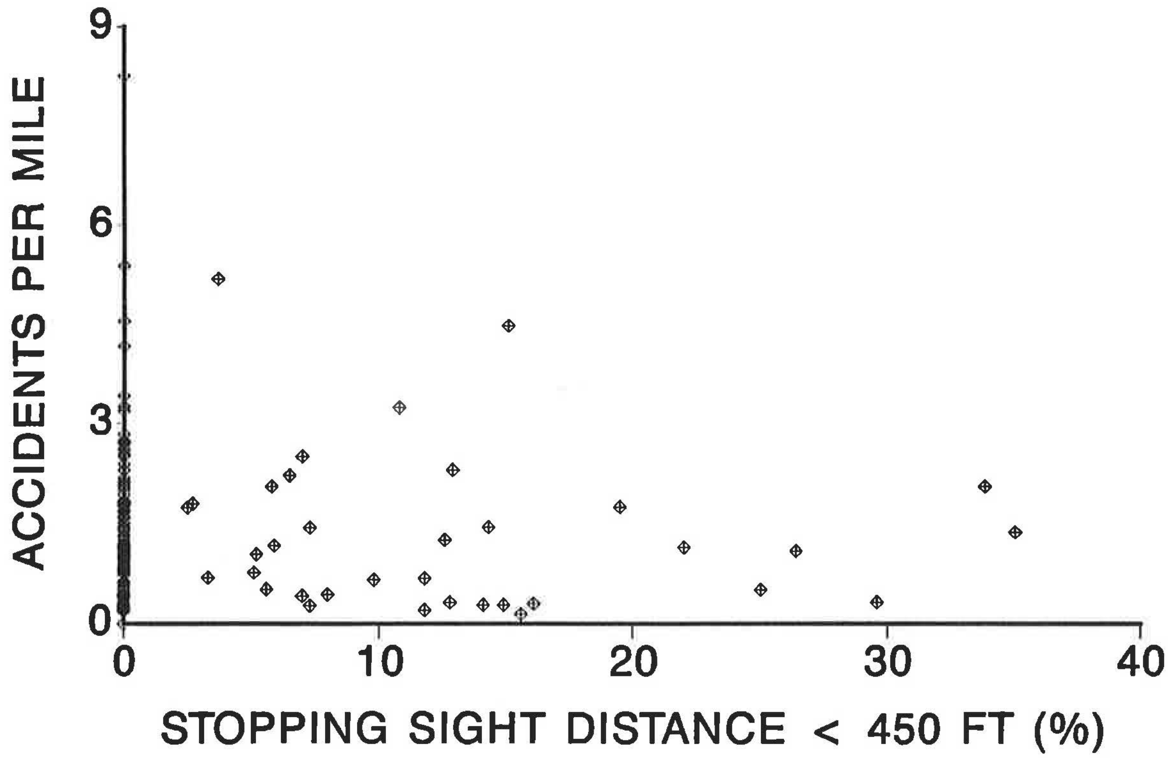

FIGURE 2 Relationship of accident rate per mile and percentage of limited SSD, two-lane roads with shoulders.

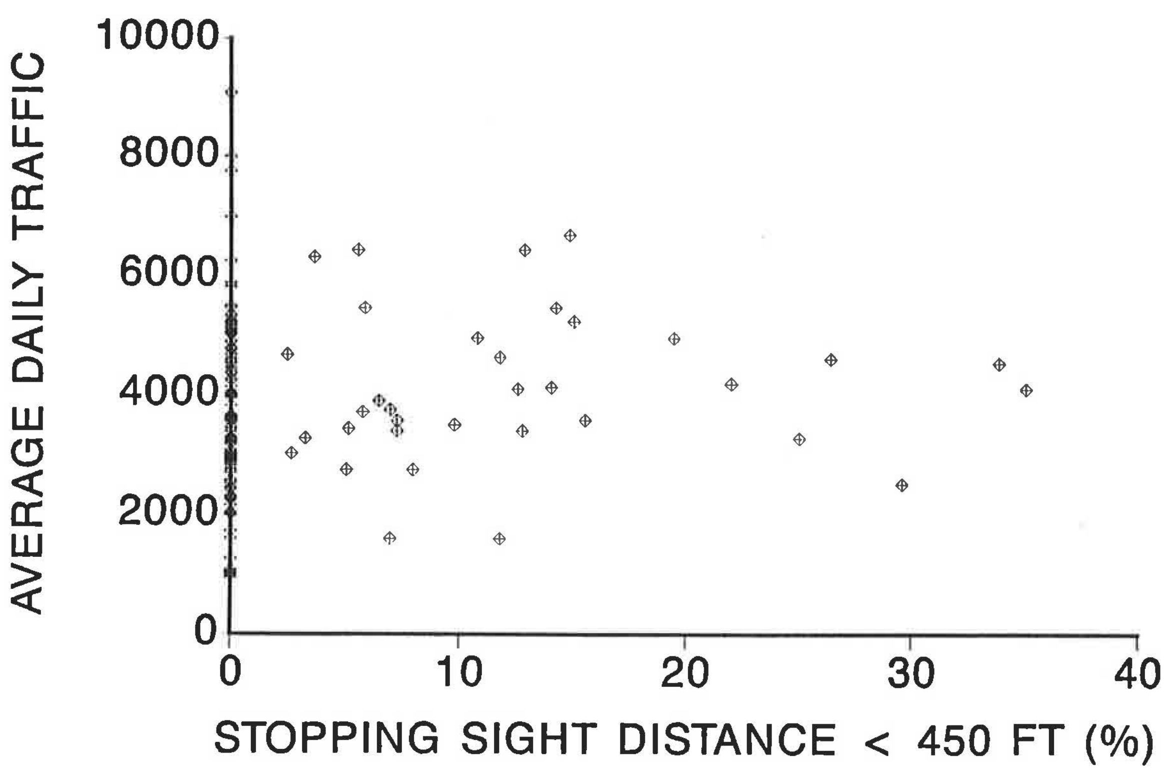

FIGURE 3 Relationship between limited SSD and AADT, two-lane roads with shoulders.

TABLE 3 INTERSECTIONS ON TWO-LANE ROADWAYS WITH SHOULDERS BY AVAILABLE SSD

Type of Intersection	Total	Number of Intersections Available SSD (Ft.)		
Road		<325	<450	<550
Numbered	53	0	4	5
County	221	0	9	22
Driveway	25	0	6	9
Total	299	0	19	36

TABLE 4 REGRESSION COEFFICIENTS FOR ANALYSIS OF MINIMUM CRITERION OF SIGHT DISTANCE FOR 55 MPH (450 FT) ON TWO-LANE ROADWAYS WITH SHOULDERS

	Dependent Variable: Accidents per mile	Logarithm of Accidents per mvm
Intercepts		
Neither County nor Numbered Roads	-0.1559	0.4065**
County Road, No Numbered Road	-0.1179	0.4258**
Numbered Road, No County Road	0.1624	0.6805**
Both County and Numbered Roads	0.1222	0.6255**
AADT	0.0002563**	0.00002317
AADT²	-0.0000000126+	----
Intersecting Roads within Influence of Restricted Sight Distance	-0.5452*	-0.4139*
Interaction of AADT and Intersection Roads	0.0001522*	0.0001169+

+ =p < .1
* =p < .05
** =p < .01

crest curves did have a significant effect when included in the model along with its interaction with AADT. The presence of the interaction with AADT yields a model with an increasing effect of the intersections as AADT increases.

Examination of the alternative dependent variable, logarithm of accidents per MVM, yielded similar results. The estimated coefficients from the two analyses are presented in Table 4. Note the negative coefficients associated with the number of curve-influenced intersecting roads within limited stopping sight distance sections. The effect of the negative coefficient is overshadowed, however, by the positive coefficient associated with the interaction of AADT with this factor. A slight negative effect on accident rates is seen at low AADT values, but an overwhelming positive effect of curve-influenced intersections is demonstrated at higher AADT values. These results are demonstrated in Table 5, which provides estimated values of accident rates from the model for accidents per mile. Note the slight decrease in accident rate as the number of intersections increases for AADT of 2,000. The effect is reversed, however, for the higher AADT values. The effects seen at the outer ranges of the data are extreme and should not be accepted casually. The more reliable estimates are associated with AADT values between 3,000 and 5,000 vehicles, which represent over half of the sample data. These estimates assume a county road, but no numbered road on the segment.

TABLE 5 ESTIMATED VALUES OF ACCIDENTS PER MILE ON TWO-LANE ROADWAYS WITH SHOULDERS

AADT	Number of Intersections Within Limited SSD Sections			
	0	1	2	3
2000	0.41	0.11	0.00	0
4000	1.02	1.16	1.30	1.45
6000	1.63	2.80	4.45	6.93

The plot of accident rate versus percent limited sight distance is repeated in Figure 4, with the sample points containing curve-influenced intersecting roads indicated. Note that the majority of such points are associated with higher accident rates. This result is what is being brought out by the regression analysis.

The same analyses were carried out using the more conservative measure of sight distance. The value of 550 ft, which is the minimum value in the AASHTO policy for 65 mph, was used to calculate the percent of limited sight distance. These analyses yielded essentially the same results as those for the 450-ft criterion for both accidents per mile and accidents per MVM. The effects of intersections within limited SSD sections were statistically significant ($p > 0.05$) in both these analyses.

Two-Lane Roads without Shoulders

A smaller sample of 54 approximately one-mile road segments was defined from the selection of two-lane roads without shoulders that was identified by the Texas State Department of Highways and Public Transportation district offices. The total number of accidents occurring on these segments was 464. Annual accident rates per mile varied between 0.0 and 7.19.

Examination of the distribution of AADT in this sample showed that there was very limited data available for AADT greater than 4,000 vehicles. Further, Table 6 provides the cross-classification of AADT and percent of limited sight distance using the AASHTO minimum criterion for 55 mph of 450 ft. The data illustrate extreme imbalance with respect to these two important variables. Only nine road segments are identified with AADT greater than 4,000, and each of these segments has little roadway with limited sight distance. Figure 5 provides the plot of the relationship and it can again be seen that the segments with the higher AADT values are

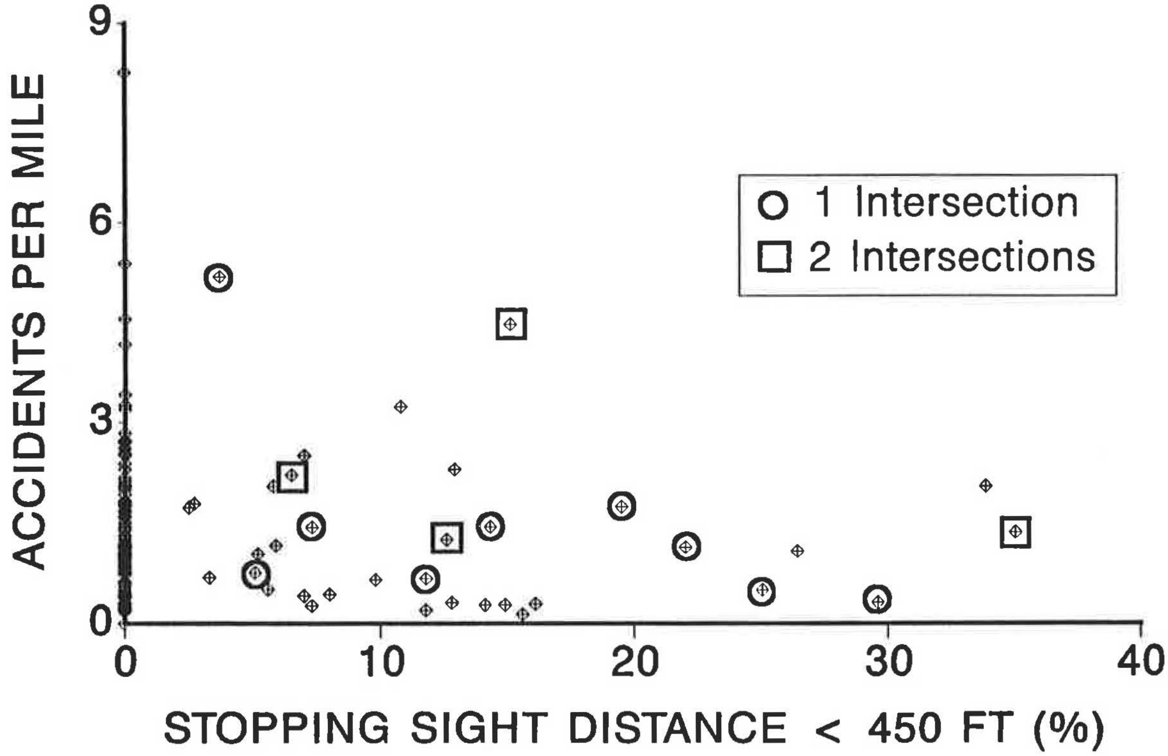

FIGURE 4 Relationship between limited SSD and accidents per mile with consideration of curve-influenced intersecting roads, two-lane roads with shoulders.

TABLE 6 FREQUENCY BY AADT AND PERCENT LIMITED SIGHT DISTANCE (450 FT) ON TWO-LANE ROADWAYS WITHOUT SHOULDERS

Percent Limited Distance	AADT			
	<2000	2-3999	4-5999	≥6000
0	3	8	3	2
1-10	4	2	0	1
11-20	6	11	1	2
21-30	1	6	0	0
31-40	0	3	0	0
>40	0	1	0	0

indeed restricted to low values of percent limited stopping sight distance. The higher AADT roadways do not contain large amounts of crest vertical curves with limited stopping sight distance.

Because of the importance of accurately adjusting for AADT before evaluating the relationship between accident rates and limited sight distance, this group of road segments was split according to AADT values before proceeding with the analysis. This step was deemed necessary because of the strong imbalance between AADT and percent limited stopping sight distance. Given the extremely unbalanced sample data, it could not be ensured that the modeling of accident rate on AADT would be adequate, and thus the evaluation of the effect of limited sight distance could be biased. The analysis could be performed in two parts, eliminating the problems just outlined. Because of the scarcity of data for AADT greater than 4,000, only those segments with AADT less than 4,000 were analyzed in order to eliminate the potential bias caused by imbalance.

This study sample of two-lane roads without shoulders represents roads with considerably more limited sight distance sections than the previously analyzed roadways with shoulders. The available information on sight distance for each of the three criteria is shown in Table 7. These frequencies are restricted to those road segments with AADT of less than 4,000. Almost all segments have limited sight distance sections when the more conservative criteria are used to define the measurement. A small percentage are, in part, limited using the stopping sight distance criterion of 325 ft.

As in the previous data set, only a small percentage of the intersecting roads are within the limited stopping sight distance sections of crest curves. Table 8 gives the breakdown of the data available on intersections within limited stopping sight distance sections for the two-lane-without-shoulders data set.

The variable for limited sight distance using the criterion of 325 ft of required sight distance is examined first. Figures 6 and 7 present the accident rate per mile against AADT and percent limited stopping sight distance, respectively. The same observations that were made previously in examining the first data set (two-lane roads with shoulders) hold here as well. In Figure 7, note the limited data available for percent stopping sight distance. The range is from 0 to less than 15

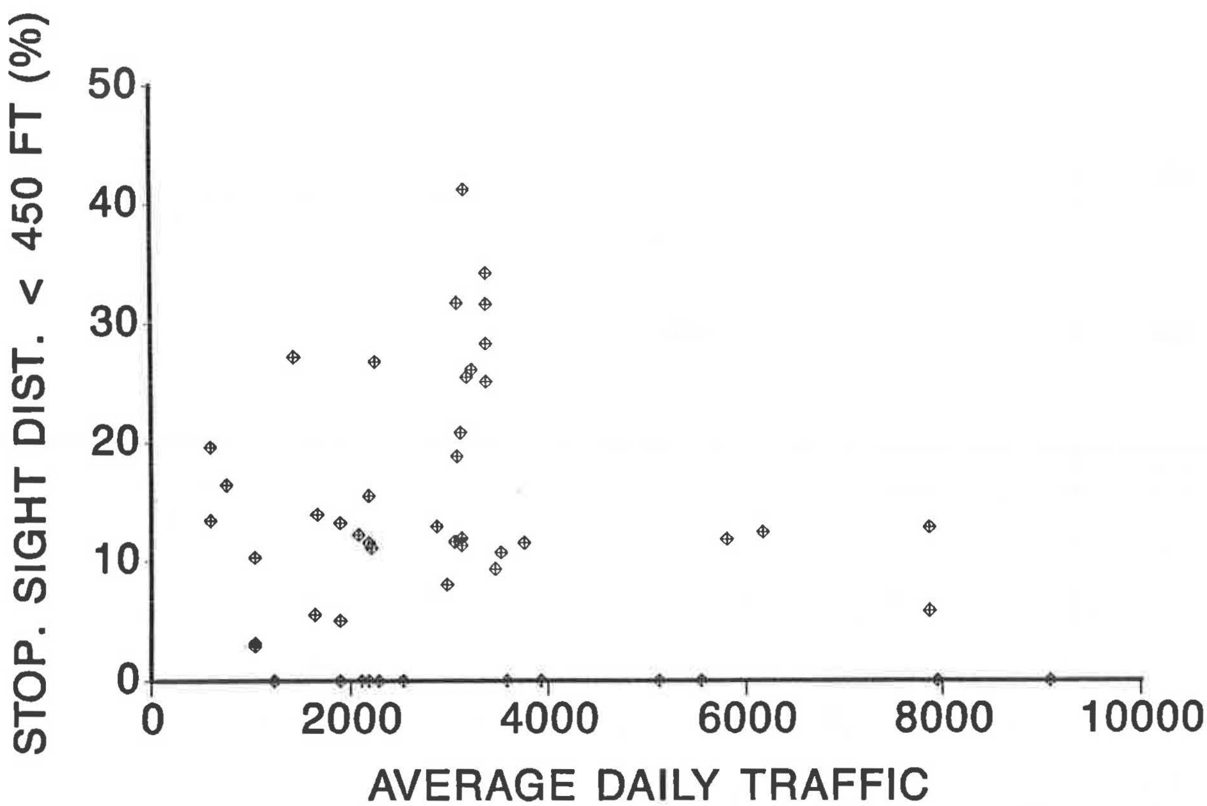

FIGURE 5 Relationship between AADT and limited SSD, two-lane roads without shoulders.

TABLE 7 FREQUENCY OF SEGMENTS BY THREE CRITERIA OF LIMITED SIGHT DISTANCE ON TWO-LANE ROADWAYS WITHOUT SHOULDERS

Percent Limited Distance	Minimum 325	Sight 450	Distance (ft) 550
0	33	11	1
1-10	9	6	2
11-20	3	17	18
21-30	0	7	10
31-40	0	3	8
41 +	0	1	6
Total	45	45	45

TABLE 8 INTERSECTIONS ON TWO-LANE ROADWAYS WITHOUT SHOULDERS BY AVAILABLE SSD

Type of Intersecting	Number of Intersections Total	Available SSD (ft)		
Road		<325	<450	<550
Numbered	13	0	3	7
County	72	0	4	9
Driveway	12	3	7	11
Total	97	3	14	27

percent. The number of intersections within limited stopping sight distance sections is indicated in this figure.

The regression analysis was performed in the same way as for the analysis of two-lane roads with shoulders. The results of the regression of the logarithm of accidents per mile as the dependent variable are presented in detail. Again, the only significant effect, after adjustment for presence of major intersections and AADT, was the number of intersections within the limited stopping sight distance sections of crest curves. The interaction of this factor and AADT was not significant ($p > 0.1$), indicating a clearly positive relationship with accident rate for all AADT values.

Figure 8 illustrates the relationship between accidents per mile and the percent limited sight distance using the 450-ft criterion. Indicated are the values associated with those segments containing intersecting roads within the limited stopping sight distance sections of curves. Again, it can be seen that the segments with the highest numbers of intersecting roads within limited stopping sight distance sections of crest vertical curves have some of the highest accident rates. Also, there appears to be a negative relationship between accident rate and the percent limited stopping sight distance.

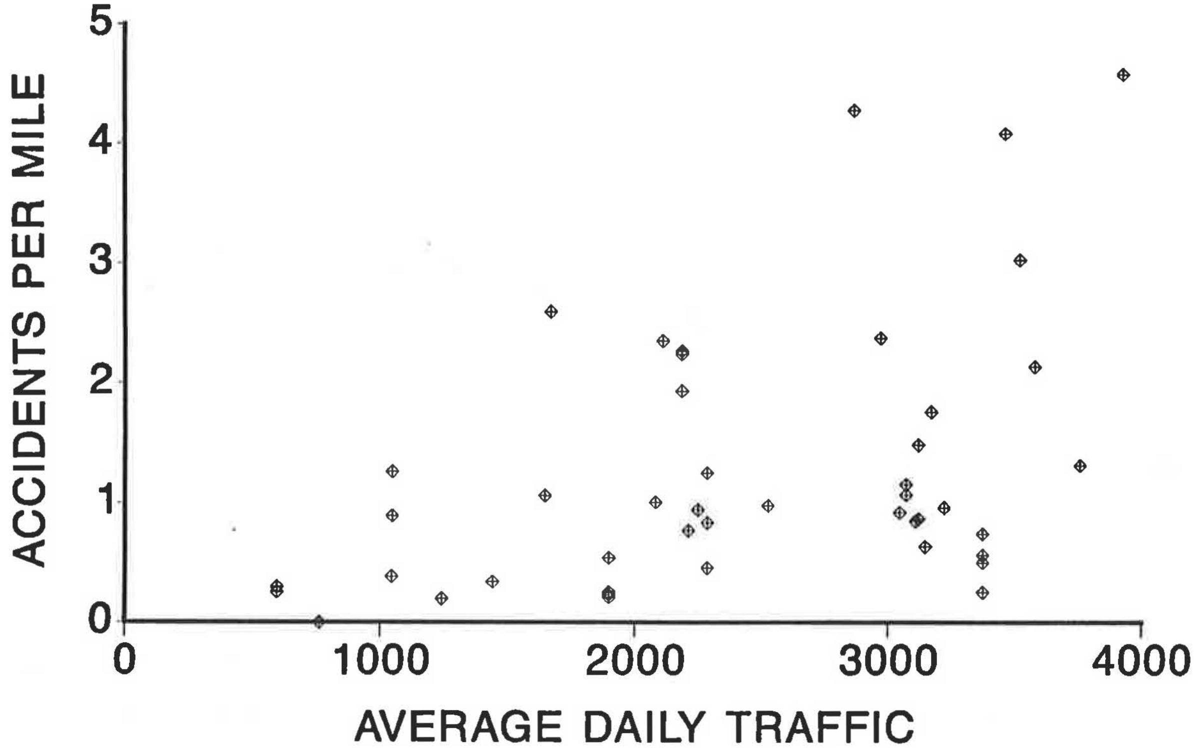

FIGURE 6 Relationship between AADT and accidents per mile with limited SSD, two-lane roads without shoulders.

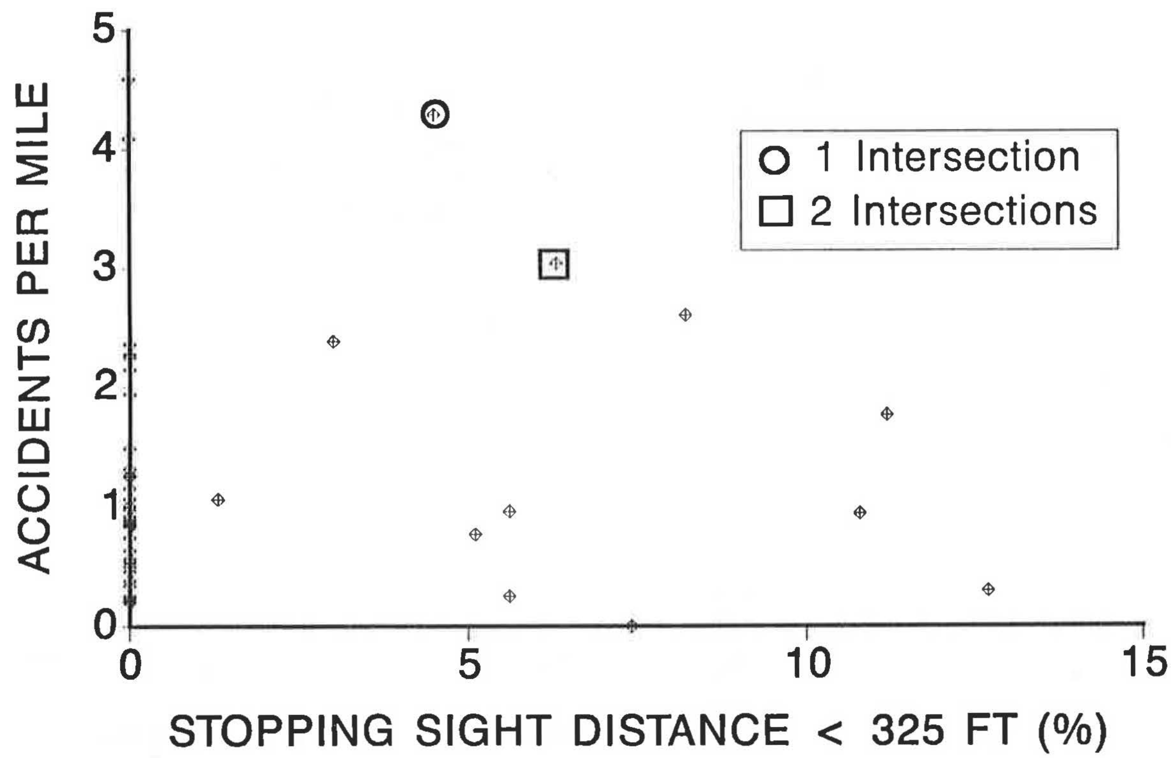

FIGURE 7 Relationship between limited SSD and accidents per mile with consideration of curve-influenced intersecting roads, two-lane roads without shoulders.

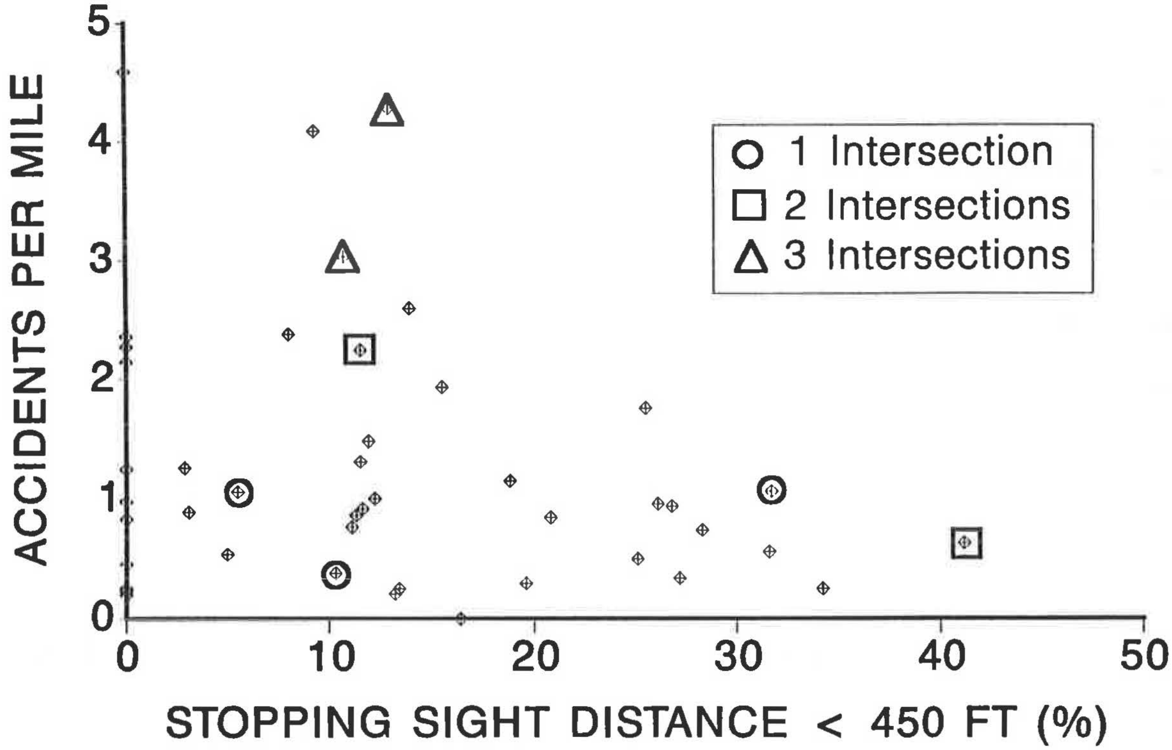

FIGURE 8 Relationship between accidents per mile with limited SSD with consideration of curve-influenced intersecting roads, two-lane roads with shoulders.

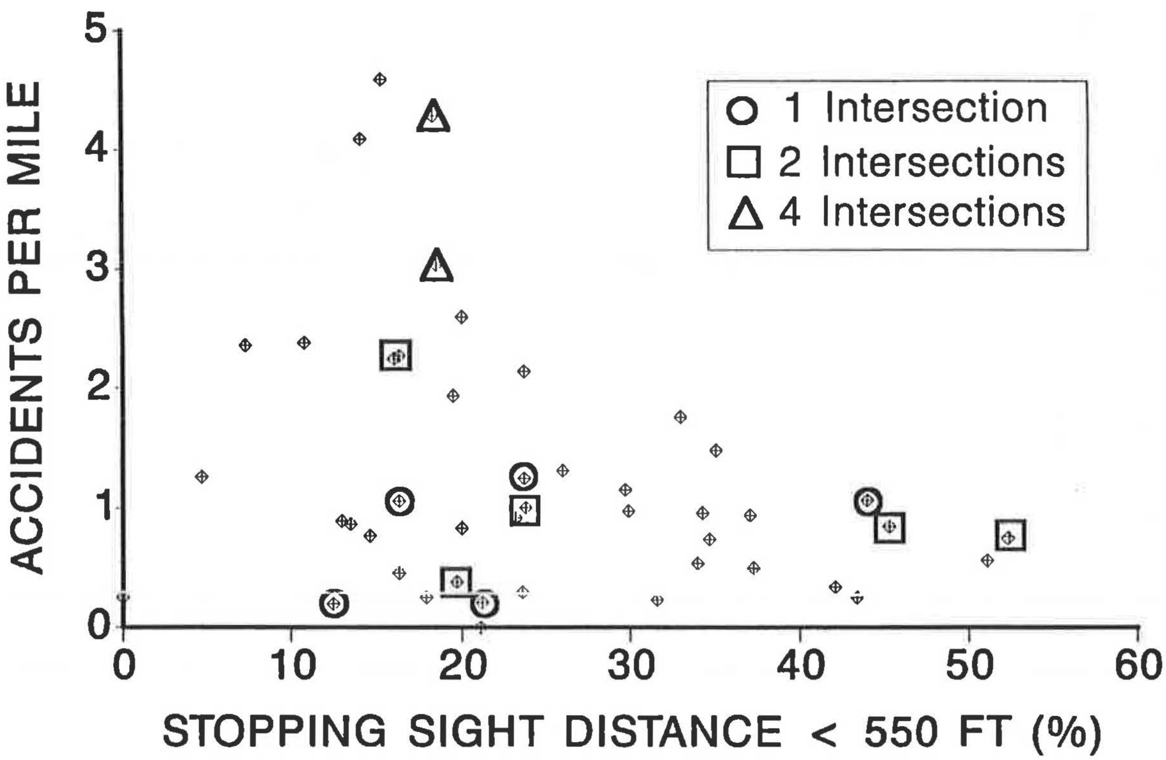

FIGURE 9 Relationship between limited SSD and accidents per mile with consideration of curve-influenced intersecting roads, two-lane roads without shoulders.

The regression analysis produced ambiguous results. The effect of the number of intersecting roads within limited stopping sight distance sections on the accident rate was positive and significant, but accompanying this effect was a significant negative relationship between accident rate and the percent of the roadway with limited sight distance.

The effect of percent limited sight distance using the most conservative standard of 550 ft was finally examined. The results of that analysis repeated the negative association between accident rate and percent limited stopping sight distance. Again, the percent limited distance was highly significant, with a negative coefficient. Figure 9 presents the results. The effect of the number of intersecting roads within limited stopping sight distance sections of crest curves was not significant (p = 0.15).

The coefficients from these three analyses are given in Table 9 for comparison. Special notice can be made of the relative sizes of the coefficients estimating the effect of the number of intersecting roads within limited stopping sight distance sections of crest curves. It is of interest that these coefficients are reduced by roughly one-half as the criterion for measuring limitations in sight distance becomes more conservative. For example, the coefficient of 0.36 for the 325-ft criterion is reduced to 0.17 when the "minimum" AASHTO criterion for 55 mph (450 ft) is used. And the size of the corresponding coefficient for the 550-ft criterion was 0.07, which was found to be nonsignificant (p = 0.15) and thus is not included in Table 9.

Both the models for the AASHTO minimum standard for 55 mph and the minimum for 45 mph contain significant coefficients for the effect of intersections within limited stopping sight distance sections, using the adopted significance level of 0.05. And the effect, if added to the model developed for 550 ft required sight distance, approached significance (p = 0.15). Nevertheless, the negative relationship between accident rates and percent limited sight distance remains for the 450- and 550-ft standards. The effect is clearly negative for stopping sight distance of 550 ft. And the effect sometimes overwhelms the positive effect of the intersections, yielding estimated accident rates that decrease for increasing percent limited distance with a stopping sight distance of 450 ft. Examples of this relationship are given in Table 10. The estimates assume the presence of a county road, but no numbered road.

TABLE 9 REGRESSION COEFFICIENTS FROM THE ANALYSIS OF LOGARITHM OF ACCIDENTS PER MILE ON TWO-LANE ROADWAYS WITHOUT SHOULDERS

	Sight Distance Criterion (ft)		
	325	450	550
Intercepts			
Neither County nor Numbered Roads	0.0986	-0.2404	0.3624+
County Road, No Numbered Road	0.3428*	-0.1079	0.4436**
Both County and Numbered Roads	0.4586+	-0.0655	0.5145*
AADT	0.0001645*	0.0004043**	0.0002929**
Percent Limited Sight Distance	------	0.02223	-0.01715**
Interaction of AADT and Percent Limited Sight Distance	------	-0.00001354*	-------
Intersecting Roads within Influence of Restricted Sight Distance	0.3592*	0.1741*	------

\+ =p < .1
* =p < .05
** =p < .01

TABLE 10 ESTIMATED ACCIDENTS PER MILE FOR SSD CRITERION—450 FEET ON TWO-LANE ROADWAYS WITHOUT SHOULDERS

Percent Limited Stopping Sight Distance	Number of Intersections Within Limited SSD Sections			
	0	1	2	3
0	1.02	-	-	-
20	0.83	1.18	1.59	2.08
40	0.66	0.98	1.35	1.80
0	3.52	-	-	-
20	1.39	1.84	2.38	3.03
40	0.26	0.50	0.79	1.13

Examination of accidents per MVM, which was an additional dependent variable for analysis, did not significantly alter any results already obtained using the accident rate per mile.

In an attempt to understand the conflicting relationships modeled in this analysis, the values that seem to have the most influence on the negative relationship between accident rate and percent limited distance were examined. It was discovered that most of the segments that contain large relative amounts of limited stopping sight distance were from one area, all belonging to the same roadway section. Figure 10 identifies these points.

All the analyses for logarithm of accidents per mile, a dependent variable, were repeated, omitting this section of roadway. Many effects were no longer significant, which may be partly attributed to the reduction in the size of the data set. The coefficients and their significance levels for the adopted models for the three criteria were already presented in Table 4. Those results can be compared with these modified analyses. For the 325-ft criterion, the variable for intersections within limited stopping sight distance sections became insignificant (p = 0.12), leaving only the types of major intersections and AADT in the model. In the analysis of the 450-ft criterion, the negative relationship between accident rate and percent limited stopping sight distance was dropped from the model because of nonsignificance, leaving only the positive relationship with intersections within limited stopping sight distance sections. The resulting model, therefore, clearly predicts an increase in accident rate as the number of intersections within limited stopping sight distance sections increases. The model for the 550-ft stopping sight distance criterion remained unchanged, although the significance level was reduced (p = 0.04).

Another method of adjusting for differences that are known to exist among roadways, but for which we have no quantifiable measurements, was used. A constant term was introduced into the model for each different roadway, distin-

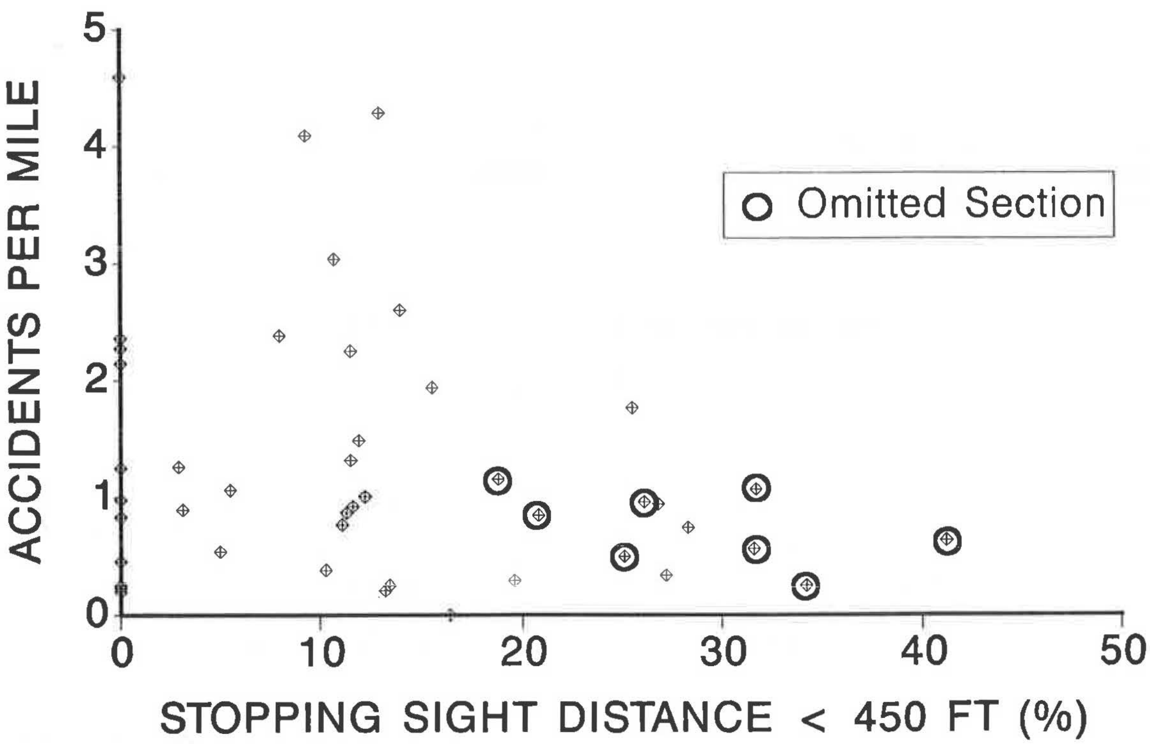

FIGURE 10 Relationship between limited SSD and accidents per mile, two-lane roads without shoulders.

guished by its control number. This method allowed an individual constant adjustment of the accident rate in each case. Again, some effects disappeared after incorporating this adjustment. Nonetheless, the positive effect of intersecting roads within limited stopping sight distance sections of crest curves based on the 325-ft criterion increased in its effect. The coefficient increased to 0.41, compared with 0.36 previously, with a significance probability of 0.01. All effects related to stopping sight distance (the relative amounts and the number of intersections within limited stopping sight distance sections) were no longer significant ($p > 0.05$) in the analyses of the 450- and 550-ft criteria.

These results are more meaningful when compared with similar analyses on the previous data set. The two-lane roads without shoulders were subjected to the same adjustment for different roadways for comparison. In the analysis of that data set, no changes in the models resulted. The models remained remarkably consistent in terms of the sizes of the coefficients as well. The limited data available for the analysis of two-lane roads without shoulders makes the ambiguous results open to question. The consistency of the analytical results of the larger sample of two-lane roads with shoulders can be interpreted with more confidence.

SUMMARY AND CONCLUSIONS

Two large data bases consisting of 222 study segments of approximately one-mile lengths representing nearly 1,500 accidents were assembled to evaluate the effects that stopping sight distance along crest vertical curves has on accident rates. The study sites were carefully screened to control for other geometric and operational conditions that could affect accident rates. All study segments were two-lane roadways. The roadways had 55 mph posted speeds and were located in rural areas of east and central Texas. The study segments with sight distance limitations generally had modest limitations; that is, sight distance limitations were generally less than the AASHTO minimum requirement for 55 mph design, but they were generally better than the AASHTO minimum requirements for a 45 mph design. The following are the most significant findings:

1. The relationship between available sight distance on crest vertical curves on two-lane roadways and accidents is difficult to quantify even when a large data base exists.
2. The AASHTO stopping sight distance design model alone is not a good indicator of accident rates on two-lane rural roadways in Texas. Thus, adherence to the model alone in designing projects will not result in cost-effective projects.
3. Where there are intersections within the limited sight distance sections of crest vertical curves, there is a statistically significant increase in accident rates.
4. It can be inferred that other geometric conditions within limited sight distance sections of crest vertical curves could also cause a marked increase in accident rates. An example would be a sharp horizontal curve hidden by a crest vertical curve.
5. The increase in accident rates because of intersections within limited sight distance sections of crest vertical curves is more pronounced on roadways with higher volumes, implying that a threshold volume level may be determined based on considerations of cost effectiveness.

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Stopping Sight Distance Design for Large Trucks

DOUGLAS W. HARWOOD, WILLIAM D. GLAUZ, AND JOHN M. MASON, JR.

Stopping distance requirements for large trucks are compared with current AASHTO stopping sight distance criteria. Key elements affecting stopping sight distance for trucks include perception-reaction time, truck braking distance, and truck driver eye height. The paper stresses the variability of truck driver braking performance and the safety benefits associated with antilock brake systems for trucks. Findings indicate that trucks with conventional brake systems may require stopping sight distances greater than those recommended by current AASHTO policy. The increased values potentially affect all related stopping sight distance design considerations (horizontal and vertical curvature, intersection sight distance, and highway-railroad grade crossings). The magnitude of increase, however, is highly dependent on individual driver brake performance capabilities. For drivers whose emergency braking performance is equivalent to the worst performance observed in braking tests for conventional brake systems, substantially greater stopping sight distance and longer vertical curves would be needed than are used under current AASHTO criteria. Drivers with braking performance equivalent to the best performance observed in braking tests for conventional brake systems require only slightly longer stopping sight distance than current AASHTO criteria and require vertical curve lengths that are shorter than current AASHTO criteria. If antilock brake systems are eventually mandated for trucks, current AASHTO stopping sight distance policy would adequately accommodate the needs of large trucks.

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

This paper examines the suitability of current stopping sight distance design criteria for large trucks in light of available data concerning truck characteristics, including braking distance and driver eye height. The paper uses the current AASHTO Green Book (*1*) stopping sight distance model as the basis for determining truck requirements. Nevertheless, the authors recognize that this model is itself in need of a thorough review to determine whether it meets the sight distance needs of drivers.

D. W. Harwood and W. D. Glauz, Midwest Research Institute, 425 Volker Blvd., Kansas City, Mo. 64110. J. M. Mason, The Pennsylvania State University, University Park, Pa. 16820.

CURRENT DESIGN CRITERIA

This section summarizes the current AASHTO design criteria for stopping sight distance.

Stopping Sight Distance Criteria

Stopping sight distance is determined in the AASHTO Green Book (*1*) as the sum of two terms: brake reaction distance and braking distance. The brake reaction distance is the distance traveled by the vehicle from the driver's first sighting of an object necessitating a stop to the instant the brakes are applied. The braking distance is the distance required to bring the vehicle to a stop once the brakes are applied.

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

$$S = 1.47\, t_{pr} V + \frac{V^2}{30f} \qquad (1)$$

where

S = stopping sight distance (ft),
t_{pr} = perception-reaction time (sec),
V = initial vehicle speed (mph), and
f = coefficient of tire-pavement friction.

The first portion of Equation 1 represents the brake reaction distance, and the second term represents the braking distance. The factors that influence braking distances are discussed later in this paper. The coefficient of sliding friction is used by AASHTO in Equation 1 to determine the braking distance for a locked-wheel stop by a passenger car.

Table 1 presents the AASHTO Green Book criteria for stopping sight distance. These criteria are based on an assumed perception-reaction time (t_{pr}) of 2.5 sec and the assumed values of speed and coefficient of friction shown in the table. The two values shown for the assumed speed, brake reaction distance, braking distance on level, and stopping sight distance represent minimum and desirable designs, respectively. The subsequent analyses in this report are based on the desirable sight distances, which are applicable to stopping by a vehicle traveling at the design speed of the highway.

TABLE 1 AASHTO CRITERIA FOR STOPPING SIGHT DISTANCE (*1*)

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

TABLE 2 CORRECTION TO AASHTO STOPPING SIGHT DISTANCE FOR GRADES (*1*)

Increase for Downgrades: Design Speed (mph)	Correction in Stopping Distance (ft) 3%	6%	9%	Decrease for Upgrades: Assumed Speed for Condition (mph)	Correction in Stopping Distance (ft) 3%	6%	9%
30	10	20	30	28	–	10	20
40	20	40	70	36	10	20	30
50	30	70	–	44	20	30	–
60	50	110	–	52	30	50	–
65	60	130	–	55	30	60	–
70	70	160	–	58	40	70	–

Correction of Stopping Sight Distance Criteria for Grades

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

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

where G equals percent grade/100 (+ for upgrade, – for downgrade. Table 2 presents the corrections to the stopping sight distance criteria for upgrades and downgrades recommended in the AASHTO Green Book.

Application of Stopping Sight Distance Criteria to Crest Vertical Curves

Vertical crests limit the sight distance of the driver. Crest vertical curves designed in accordance with the AASHTO criteria should provide stopping sight distance at least equal to the requirements of Table 1 at all points along the curve. The minimum length of a crest vertical curve, as a function of stopping sight distance (S), is calculated by AASHTO as follows:

For S less than L_{min},

$$L_{min} = \frac{AS^2}{100\,(\sqrt{2H_e} + \sqrt{2H_o})^2} \tag{3}$$

For S greater than L_{min},

$$L_{min} + 2S - \frac{200\,(\sqrt{H_e} + \sqrt{H_o})^2}{A} \tag{4}$$

where

L_{min} = minimum length of vertical curve (ft),
S = stopping sight distance (ft),
A = algebraic difference in percent grade,
H_e = height of driver's eye above roadway surface (ft), and
H_o = height of object above roadway surface (ft).

TABLE 3 MINIMUM VERTICAL CURVE LENGTHS (IN FEET) NEEDED TO PROVIDE AASHTO STOPPING SIGHT DISTANCE

Algebraic Difference in Percent Grade	Design Speed (mph) 20	30	40	50	60	70
2	60	90	150	260	610	1,070
4	60	120	300	650	1,220	2,130
6	60	170	450	970	1,820	3,190
8	70	240	600	1,280	2,420	4,260
10	90	290	740	1,610	3,030	5,320

Note: Based on AASHTO driver eye height of 42 in for a passenger car.

Equations 3 and 4 are based on the geometric properties of a parabolic curve. The AASHTO Green Book also suggests that it is typical practice to use a minimum vertical curve length that is at least three times the value of the design speed (expressed in mph). For stopping sight distance, the driver eye height (H_e) used by AASHTO is 3.5 ft, and the object height (H_o) used is 6 in. Table 3 presents the minimum vertical curve lengths required to attain the desirable stopping sight distance criteria in Table 1 as a function of design speed.

Application of Stopping Sight Distance Criteria to Horizontal Curves

Sight distance can also be limited by obstructions on the inside of horizontal curves, such as trees, buildings, retaining walls, and embankments. Horizontal curves designed in accordance with the AASHTO Green Book would provide sight distance at least equal to the requirements of Table 1 along the entire length of the curve. For a circular horizontal curve, the line of sight is a chord of that curve, and the sight distance is measured along the centerline of the inside lane. The minimum offset to a horizontal sight obstruction at the center of the curve (known as the middle ordinate of the curve) is computed in accordance with the following equation:

$$M = R\left(1 - \cos\frac{28.65\ S}{R}\right) \qquad (5)$$

where

M = middle ordinate of curve (ft),
R = radius of curve (ft), and
S = stopping sight distance (ft).

CRITIQUE OF CURRENT DESIGN CRITERIA

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

Table 4, prepared by Glennon (*2*), summarizes the historical evolution of the AASHTO stopping sight distance criteria. The Glennon summary addresses the following aspects of stopping sight distance criteria:

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

Each of these factors is discussed below.

Assumed Speed for Design

The assumed speed for stopping sight distance design has historically been less than the design speed of the highway, because it was assumed that drivers travel more slowly on wet pavements than on dry pavements. This assumption was used to derive the lower value of stopping sight distance in Table 1. AASHTO notes that recent data have shown that drivers travel about as fast on wet pavements as they do on dry pavements. Therefore, the higher values of stopping sight distance in Table 1 are based on braking by a vehicle traveling at the design speed of the highway. All analyses of stopping sight distance in this paper have been conducted with the assumption that the braking vehicle—passenger car or truck—is initially traveling at the design speed of the highway.

Brake Reaction Time

The AASHTO criteria for stopping sight distance are based on a brake reaction time of 2.5 sec. This choice for brake reaction time has been confirmed as appropriate for most drivers by a number of studies, including, most recently, Johansson and Rumar (*7*) and Olson et al. (*8*).

The brake reaction time is a driver characteristic and is assumed to be applicable to truck drivers as well as passenger car drivers, although experienced professional truck drivers could reasonably be expected to have shorter brake reaction times than the driver population as a whole. Nevertheless, the air brake systems commonly used in tractor-trailer combination trucks have an inherent delay of approximately 0.5 sec in brake application (*9*). For purposes of this analysis, it is assumed that these factors offset one another and that the 2.5-sec brake reaction time is appropriate for trucks.

Coefficient of Tire-Pavement Friction

The coefficients of friction shown in Table 1 were chosen from the results of several studies cited in Figure III-1 of the AASHTO Green Book, and they are intended to represent the deceleration rates used by a passenger car in locked-wheel braking on a poor, wet pavement. The results cited by AASHTO that most closely match the criteria in Table 1 are from a 1951 study by Moyer and Shupe (*10*) and are based on locked-wheel skid test results obtained for new passenger car tires.

An exceedingly important feature of truck stopping distance is that trucks cannot safely make a locked-wheel stop without the risk of losing control of the vehicle. The discussion of braking distances later in this paper shows that the deceleration rates used by trucks in making controlled stops are

TABLE 4 EVOLUTION OF AASHTO STOPPING SIGHT DISTANCE POLICY (2)

	DESIGN PARAMETERS					
YEAR	Eye Height (feet)	Object Height (Inches)	Perception/ Reaction Time (Seconds)	ASSUMED TIRE/PAVEMENT COEFFICIENT OF FRICTION	ASSUMED SPEED FOR DESIGN	EFFECTIVE CHANGE FROM PREVIOUS POLICY
1940 (3)	4.5	4	Variable-- 3.0 Sec. @ 30 mph to 2.0 Sec. @ 70 mph	DRY-- f Ranges from 0.50 @ 30 mph to 0.40 @ 70 mph	DESIGN SPEED	--
1954 (4)	4.5	4	2.5	WET-- f Ranges from 0.36 @ 30 mph to 0.29 @ 70 mph	Lower Than Design Speed (28 mph @ 30 mph Design Speed; 59 mph @ 70 mph Design Speed)	No Net Change in Design Distances
1965 (5)	3.75	6	2.5	WET-- f Ranges from 0.36 @ 30 mph to 0.27 @ 80 mph	Lower Than Design Speed (28 mph @ 30 mph Design Speed; 64 mph @ 80 mph Design Speed)	No Net Change in Design Distances
1971 (6)	3.75	6	2.5	WET-- f Ranges from 0.35 @ 30 mph to 0.27 @ 80 mph	Minimum Values-- Same as 1965; Desirable Values-- DESIGN SPEED	Desirable Values are up to 250 feet greater than minimum values
1984 (1)	3.50	6	2.5	WET-- f slightly lower than 1970 values for higher speeds	Minimum Values-- Same as 1965; Desirable Values-- DESIGN SPEED	Computed values always rounded up giving slightly higher values than 1970

generally lower than the deceleration rates used by passenger cars making locked-wheel stops.

Driver Eye Height

The minimum crest vertical curve criteria for stopping sight distance in Table 3 are based on a driver eye height for passenger cars of 3.5 ft (42 in). The driver eye heights for trucks are much greater than for passenger cars, which may partially or completely offset their longer braking distances on crest vertical curves. Nevertheless, the greater eye heights of truck drivers provide no comparable advantage for sight obstructions on horizontal curves unless the truck driver is able to see over the obstruction.

A review of recent evaluations of truck driver eye height, including studies by Middleton et al. (*11*), Burger and Mulholland (*12*), and Urban Behavioral Research Associates, Inc. (*13*), found that truck driver eye heights can range from 71.5 to 112.5 in. Middleton et al. estimated the average driver eye height for a conventional tractor to be 93 in. This value was also used by Olson et al. (*8*) in their recent studies of stopping sight distance. Nevertheless, it is important to remember that 93 in represents an average truck driver eye height and that below average eye heights should also be considered. This paper includes sensitivity analyses for truck driver eye heights of 75 and 93 in.

Object Height

The object height used in determining the crest vertical curve lengths in Table 3 is 6 in. As shown in Table 4, a 4-in object height was used prior to 1965. The AASHTO Green Book presents the object height as an arbitrary rationalization of possible hazardous objects that could be found in the roadway. Others maintain that object height has historically represented a subjective tradeoff of the cost of providing sight distance to the pavement and did not represent any particular hazard (*2*). The recent analysis of this issue by Olson et al. (*8*) assumed that the object height was meant to represent a specific possible hazard, but questioned the use of a 6-in object based on a study by Woods (*14*), which found that about 30 percent of compact and subcompact passenger cars could not clear an object of that height. Whatever interpretation of object height is chosen, the crest vertical lengths for trucks should not be affected because trucks typically have underclearances substantially greater than 6 in.

Horizontal Sight Obstructions

Increased eye height provides truck drivers no advantage over passenger car drivers at a horizontal sight obstruction, unless the truck driver is able to see over the obstruction. Nevertheless, Olson et al. (*8*) indicate that the minimum offset to a horizontal sight obstruction (represented by the middle ordinate of the curve computed with Equation 5) is normally required only near the center of a horizontal curve. Figure 1 illustrates a sight distance envelope—or "clear sight zone"—where horizontal sight obstructions should not be present. The figure illustrates that less than the maximum offset to horizontal sight obstructions is needed within a distance to either end of the curve equal to half of the stopping sight distance.

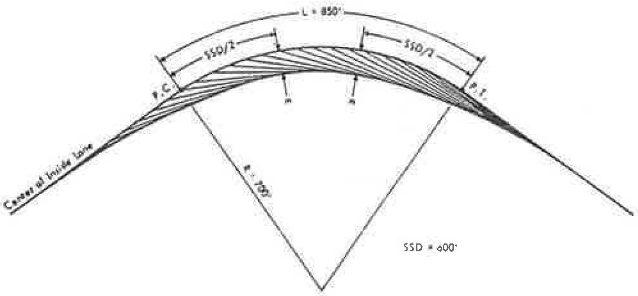

FIGURE 1 Example sight obstruction envelope on horizontal curves for condition where the stopping sight distance is less than the length of the curve.

Another problem associated with stopping sight distance on horizontal curves cited by Olson et al. (*8*) and Neuman et al. (*15*) is that the tire-pavement friction available for braking is reduced by the portion of the available tire-pavement friction that is required for cornering. Olson et al. express the available friction for braking on a horizontal curve as

$$f^2 = f_t^2 - \left(\frac{V^2}{15R} - e\right)^2 \tag{6}$$

where

f = coefficient of friction available for braking,
f_t = total available coefficient of friction,
V = vehicle speed (mph),
R = radius of curvature (ft), and
e = superelevation rate (ft/ft).

Equation 6 implies that the required stopping sight distances on horizontal curves should be longer than on tangents.

TRUCK BRAKING DISTANCE

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

Tire-Pavement Friction in Braking Maneuvers

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

The coefficient of braking friction (f_y) is defined as the ratio of the braking force (F_y) generated at the tire-pavement interface to the vertical load (F_z) carried by the tire. This can be expressed as

$$f_y = \frac{F_y}{F_z} \tag{7}$$

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

$$f_x = \frac{F_x}{F_z} \tag{8}$$

Figure 2 illustrates that both braking and cornering friction vary as a function of percent slip, which is the percent decrease in the angular velocity of a wheel relative to the pavement surface as a vehicle brakes. A freely rolling wheel is operating at 0 percent slip. A locked wheel is operating at 100 percent slip, with the tire sliding across the pavement. Figure 2 shows that the coefficient of braking friction increases rapidly with percent slip to a peak value that typically occurs between 10 and 15 percent slip. The coefficient of braking friction then

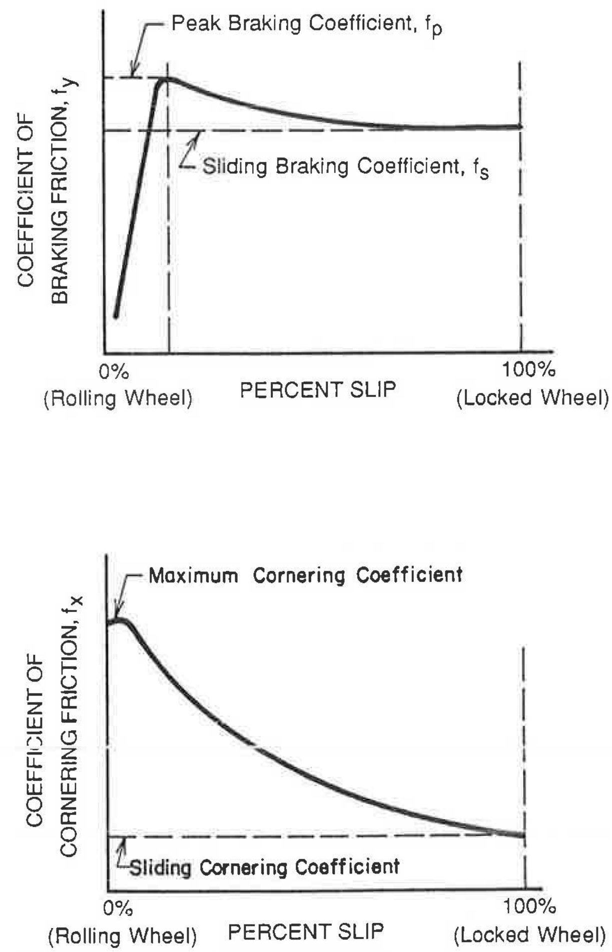

FIGURE 2 Variation of braking and cornering friction coefficients with percent slip.

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 a maximum value at 0 percent slip and decreases to a minimum at 100 percent slip. Thus, when a braking vehicle locks its wheels, it will lose its steering capability because of a lack of cornering friction.

Locked-Wheel Braking versus Controlled Braking

The discussion of Figure 2 implies that braking maneuvers can be performed in two ways: locked-wheel braking and controlled braking. Locked-wheel braking occurs when the brakes grip the wheels tightly enough to cause them to stop rotating, or "lock," before the vehicle has come to a stop. Braking in this mode causes the vehicle to slide over the pavement surface on its tires. Locked-wheel braking uses sliding friction (100 percent slip), represented by the right end of the graph in Figure 2, rather than rolling or peak friction. The sliding coefficient of friction takes advantage of most of the friction available from the pavement surface, but it is generally less than the peak available friction. On dry pavements, the peak coefficient of friction is relatively high, with very little decrease in friction at 100 percent slip. On wet pavements, the peak friction is lower, and the decrease in friction at 100 percent slip is generally larger.

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

$$BD = \frac{V^2}{30f_s} \qquad (9)$$

where

BD = braking distance (ft),
V = initial speed (mph), and
f_s = coefficient of sliding friction.

The coefficient of sliding friction in Equation 9 is mathematically equivalent to the deceleration rate used by the vehicle expressed as a fraction of the acceleration of gravity (g or 32.2 ft/sec^2). The coefficient of friction, and thus the deceleration rate, may vary as a function of speed during the stop, so f_s in Equation 9 should be understood as the average coefficient of friction or average deceleration rate during the stop.

Controlled braking is the application of the brakes in such a way that the wheels continue to roll without locking while the vehicle is decelerating. Drivers generally achieve controlled braking by "modulating" the brake pedal to vary the braking force and to avoid locking the wheels. Controlled braking distances are governed by the rolling coefficient of friction, which occurs at a value of percent slip to the left of the peak available friction shown in Figure 2. Because of the steep slope of the braking friction curve to the left of the peak and the braking techniques used by drivers to avoid wheel lockup, the average rolling friction attained is generally less than the sliding friction coefficient. Therefore, controlled braking distances are usually longer than locked-wheel braking distances.

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

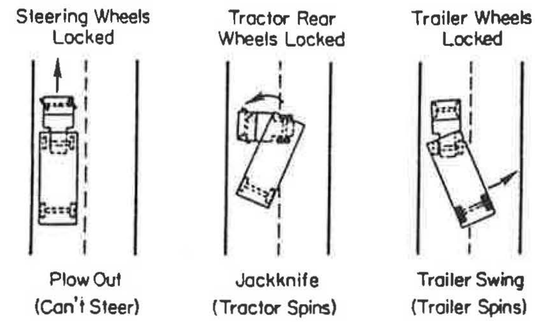

FIGURE 3 Tractor-trailer dynamics with locked wheels (*16*).

Trucks, in contrast, have much more difficulty stopping in the locked-wheel mode. Figure 3 illustrates the different dynamic responses of a tractor-trailer truck if its wheels are locked during emergency braking (*16*). The response depends on which axle is the first to lock—they usually do not all lock together. When the steering wheels (front axle) are locked, steering control is eliminated, but the truck maintains rotational stability. If the rear wheels of the tractor are locked, the axle(s) slides and the tractor rotates or spins, resulting in a "jackknife" loss of control. If the trailer wheels are locked, those axles will slide and the trailer will rotate out from behind the tractor, which also leads to loss of control. Although a skilled driver can recover from the trailer swing through quick reaction, the jackknife situation is generally not correctable. None of these locked-wheel stopping scenarios for trucks is considered safe. Therefore, it is essential that trucks stop in a controlled braking mode and that highway design and operational criteria recognize the longer distances required for trucks to make a controlled stop.

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

$$BD = \frac{V^2}{30f_r} \qquad (10)$$

where f_r equals the coefficient of rolling friction. As in the case of sliding friction, the coefficient of rolling friction (f_r) in Equation 10 represents the average coefficient of friction or average deceleration rate during the entire controlled stop.

Recent Research on Truck Braking Distance

In research at the University of Michigan Transportation Research Institute (UMTRI), Olson et al. (*8*) suggested a model to predict braking distance as a function of pavement surface characteristics, tire characteristics, vehicle braking performance, and driver control efficiency. Parametrically, the model expresses the coefficient of rolling friction, f_r, as

$$f_r = f_p \times TF \times BE \times CE \qquad (11)$$

where

f_p = peak braking friction coefficient available given the pavement surface characteristics,

TF = adjustment factor for tire tread depth (*8*),
BE = adjustment factor for braking efficiency (the efficiency of the braking system in using the available friction, typically 0.55 to 0.59 for conventional braking systems), and
CE = adjustment factor for driver control efficiency (the efficiency of the driver in modulating the brakes to obtain optimum braking performance, typically 0.62 to 1.00 for conventional braking systems).

A paper by Fancher (*17*), derived from the study by Olson et al. (*8*), used the model in Equation 11 to predict truck braking distances. Figure 4 shows the braking distances for trucks under controlled and locked-wheel stops with new and worn (2/32-in tread depth) tires in comparison with the braking distances assumed in the AASHTO Green Book. The braking distances predicted by Fancher are substantially longer than the distances for locked-wheel braking by a passenger car assumed by AASHTO. Figure 4 is based on a pavement with a skid number of 28 at 40 mph (SN_{40}) and a driver who uses 100 percent of the vehicle braking capability. Most truck drivers would require even longer stopping distances.

The data show that the braking performance of truck drivers under emergency conditions may vary widely. Most truck drivers have little or no practice in emergency braking situations. This lack of expertise in modulating of the brakes in emergency situations results in braking distances that are longer than the vehicle capability. Olson et al. (*8*) studied 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 control efficiencies ranged from 62 to 100 percent of the vehicle capability. The braking performance of the drivers tended to improve during the testing period as the drivers gained experience in modulating the brakes. Because so many drivers on the road lack experience in emergency braking, the Olson study recommended the use of a driver efficiency of 62 percent in stopping sight distance design criteria. It should be recognized that this is a very conservative choice. Experienced drivers can operate at efficiencies approaching 100 percent. Furthermore, in the future, antilock brake systems could eliminate the concern over driver efficiency by providing computer-controlled modulation of the brakes to achieve minimum braking distance.

Because truck drivers exhibit such a range of emergency braking performance, a sensitivity analysis of stopping sight distance requirements to truck driver braking performance is presented in this paper. The driver with the worst performance in this sensitivity analysis is assumed to utilize 62 percent of the vehicle braking capability (that is, CE = 0.62 in Equation 11). The driver with the best performance is assumed to utilize 100 percent of the vehicle braking capability (that is, CE = 1.0).

Figure 5 illustrates the deceleration rates (values of f_r) used to develop Figure 4. Figure 6 shows that the deceleration rates for controlled stops on a wet pavement by the best-performing driver (CE = 1.0) are generally between 0.20 and 0.25 *g*, and that they are relatively insensitive to vehicle speed. In contrast, Appendix B of the report by Olson et al. shows deceleration rates as high as 0.5 *g* in controlled stops on a wet pavement by experienced drivers. These tests were performed at the Chrysler Proving Ground on a pavement that apparently has a very high peak friction coefficient even when wet. The data in Figures 4 and 5 were derived theoretically from the model given in Equation 11.

Antilock Brake Systems

During the mid-1970s, regulations for truck braking distances were adopted, which resulted in the introduction of antilock brake systems on trucks. Shortly afterwards the restrictions were removed by court order, and because of a lack of consumer interest, trucks equipped with antilock brakes were no longer commercially available from domestic truck manufacturers. Since that time, with technological advancements and

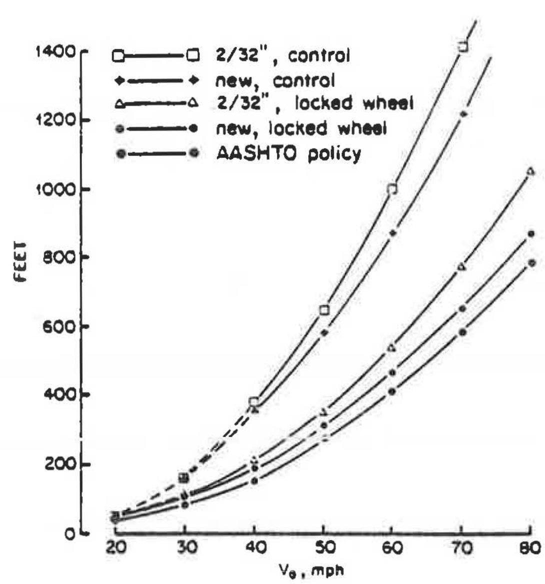

FIGURE 4 Truck braking distances on a poor, wet road (*17*).

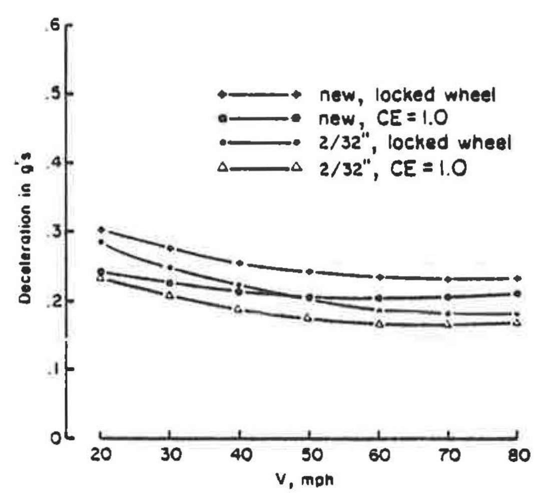

FIGURE 5 Truck deceleration rates on a poor, wet road (*17*).

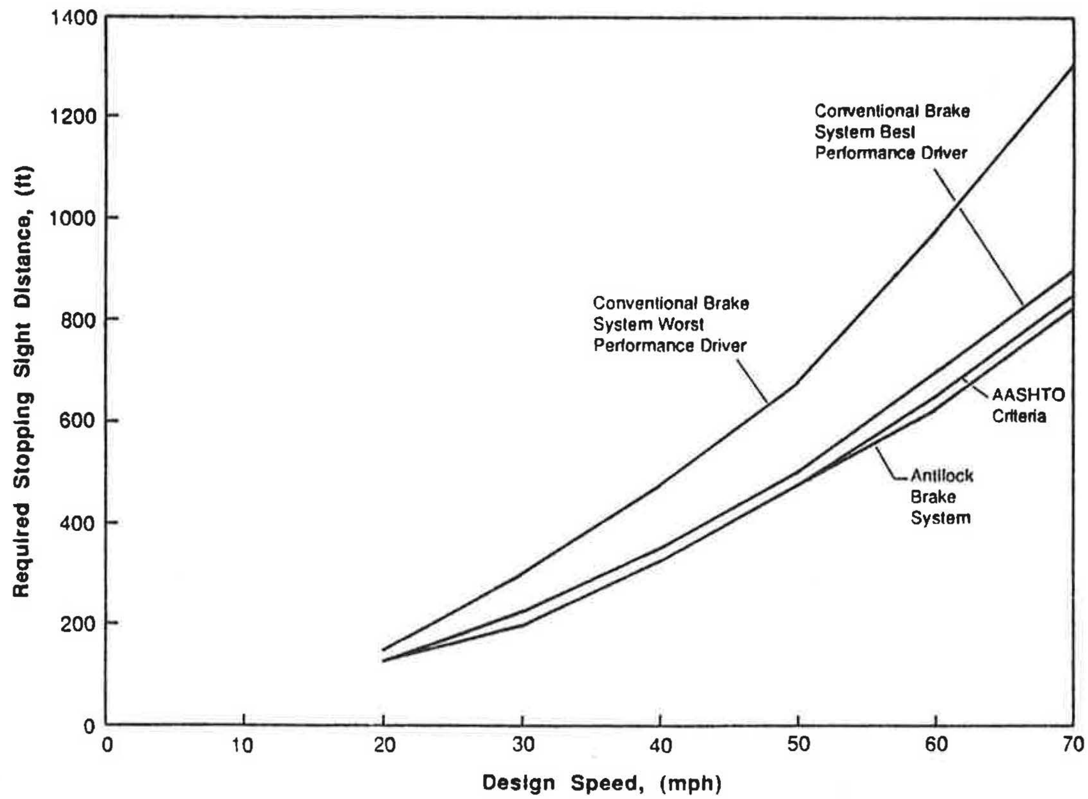

FIGURE 6 Comparison of stopping sight distance requirements for trucks with current AASHTO criteria.

improved design, antilock braking systems have gained acceptance in Europe and are slowly being reintroduced into the United States, primarily through imported passenger cars. It is possible that antilock brake systems for trucks will become common in the United States (or may be required by regulation) within 5 to 10 years. Thus, the improvements in truck braking distances that might result from antilock brake systems need to be considered in the development of future highway design criteria.

The purpose of antilock brakes is to take the fullest advantage of available tire-pavement friction capabilities without locking the wheels and losing vehicle control. Antilock brake systems are designed to achieve and maintain the peak coefficient of tire-pavement friction shown in Figure 2, maximizing the braking effect.

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

A recent NHTSA study (*18*) of the performance of a commercially available antilock brake system on a two-axle, single-unit truck found a 15 percent reduction in braking distance for a straight line stop from 60 mph on a wet, polished concrete pavement surface with an SN_{40} of approximately 30 (similar to the surface used by the AASHTO Green Book in the specification of stopping sight distance standards). Tests on other pavement surfaces and in other types of maneuvers found decreases in braking distance up to 42 percent with the antilock brake system. Furthermore, in addition to improving the braking efficiency by operating closer to the peak braking friction coefficient, antilock brake systems should also minimize the increase in braking distance caused by driver inexperience.

Design Values for Truck Braking Distance

The literature does not provide a clear indication of which braking distances should be used in highway design criteria. Many of the factors that influence braking distances, such as pavement characteristics and driver efficiencies, vary widely. For purposes of the evaluation of current highway design and operational criteria in this paper, three braking scenarios have been presented for consideration in the development of design criteria for trucks. These three scenarios are tractor-trailer truck with a conventional brake system and the worst-performing driver; tractor-trailer truck with a conventional brake system and the best-performing driver; and a tractor-trailer truck with an antilock brake system. Deceleration rates and braking distances for these three scenarios are shown in Table 5. These data are based on the results obtained by Fancher (*17*) and shown in Figures 4 and 5, with a minor change in the assumption concerning pavement surface properties (from SN_{40} of 28 assumed by Fancher to SN_{40} of 32 assumed by the AASHTO Green Book). All of the braking distances in Table 5 are appropriate for an empty truck with relatively good

TABLE 5 TRUCK DECELERATION RATES AND BRAKING DISTANCES FOR USE IN HIGHWAY DESIGN[a]

Vehicle Speed (mph)	Deceleration Rate (g) AASHTO Policy	Deceleration Rate (g) Worst Performance Driver[b]	Deceleration Rate (g) Best Performance Driver[c]	Deceleration Rate (g) Antilock Brake System	Braking Distance (ft) AASHTO Policy	Braking Distance (ft) Worst Performance Driver[b]	Braking Distance (ft) Best Performance Driver[c]	Braking Distance (ft) Antilock Brake System
20	0.40	0.17	0.28	0.36	33	77	48	37
30	0.35	0.16	0.26	0.34	86	186	115	88
40	0.32	0.16	0.25	0.31	167	344	213	172
50	0.30	0.16	0.25	0.31	278	538	333	269
60	0.29	0.16	0.26	0.32	414	744	462	375
70	0.28	0.16	0.26	0.32	583	1,013	628	510

[a] Based on an empty tractor-trailer truck on a wet pavement with SN_{40} = 32.
[b] Based on driver control efficiency of 0.62.
[c] Based on driver control efficiency of 1.00.

radial tires (at least $^{12}/_{32}$ in of tread depth). The braking distances for empty trucks are generally longer than braking distances for loaded trucks because truck brake systems are adjusted to be most effective when the truck is loaded. The braking distances in Table 5 are based on the assumption that the front-axle brakes of the truck are operational and have no automatic limiting valve.

The data for the worst-performing driver in Table 5 are based on an assumed 62 percent driver control efficiency (*CE* in Equation 11), which represents a very conservative, worst-case condition. The data for an experienced driver are based on a driver control efficiency of 100 percent and thus represent the full capability of conventional brake systems. Most truck drivers on the road today have control efficiencies that fall between these two extremes. The data for an antilock brake system represent deceleration rates between 0.31 and 0.36 *g*, which are consistent with the results of recent NHTSA tests. These estimates for antilock brake systems represent an improvement of 20 to 30 percent over the best-performing driver with a conventional brake system. The available NHTSA data (*18*) show this to be a conservative estimate of the improvement that could be obtained from future antilock brake systems.

It is important to note that the estimates of deceleration rate and braking distances in Table 5 for trucks equipped with antilock brake systems are very similar to the AASHTO criteria for passenger cars, which are also shown in the table.

SENSITIVITY ANALYSES

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

Stopping Sight Distance

Stopping sight distance criteria for trucks were derived using the AASHTO stopping sight distance relationship given in Equation 1. The stopping sight distance criteria for trucks were based on the same brake reaction time (t_{pr}) as the AASHTO criteria. The design speed of the highway is used as the initial vehicle speed in the braking maneuver. Three cases are considered for the coefficients of friction or deceleration rates used by truck drivers for controlled stops: a truck with a conventional braking system and the worst-performing driver, a truck with a conventional braking system and the best-performing driver, and a truck with an antilock brake system. The estimated deceleration rates for these three cases, shown in Table 5, are based on braking on a poor, wet road by an empty tractor-trailer truck with good tires.

Table 6 presents the stopping sight distance requirements for trucks derived from the data discussed above in comparison with the current AASHTO criteria. This comparison is also illustrated in Figure 6. Table 6 and Figure 6 show that the worst-performing driver with a conventional braking system requires substantially more stopping sight distance than the AASHTO criteria, up to 425 ft more sight distance for a 70-mph design speed. The stopping sight distance requirements for the best-performing driver with a conventional braking system are only slightly higher than the current AASHTO criteria. Thus, the assumption made about the braking performance capability, or braking control efficiency, of the driver is critical to stopping sight distance. There are essentially no data available to indicate the actual distribution of braking control efficiencies for working truck drivers.

Table 6 also shows that the sight distance requirements for

TABLE 6 STOPPING SIGHT DISTANCE REQUIREMENTS FOR TRUCKS IN COMPARISON WITH CURRENT AASHTO CRITERIA

Design Speed (mph)	Required Stopping Sight Distance (ft): AASHTO Criteria	Controlled Braking[a]: Worst Performance Driver	Controlled Braking[a]: Best Performance Driver	Controlled Braking[a]: Antilock Brake System
20	125	150	125	125
30	200	300	250	200
40	325	500	375	325
50	475	725	525	475
60	650	975	700	600
70	850	1,275	900	775

[a] Based on deceleration rates and braking distances presented in Table 5.

trucks with antilock brakes are essentially equivalent to the current AASHTO criteria. Thus, the possibility of future government requirements for truck antilock brake systems (or projected market penetration of such systems in the absence of government requirements) is critical to the assessment of stopping sight criteria. If antilock brake systems do come into fairly universal use and achieve the performance projected in Table 5, the current AASHTO stopping sight distance criteria should be adequate for trucks.

Crest Vertical Curve Lengths

Table 7 shows the minimum vertical curve lengths for a range of design speeds and algebraic differences in grade based on the stopping sight distance requirements for trucks in Table 6. The vertical curve lengths in Table 7 are based on a 6-in object height and truck driver eye heights of 75 and 93 in.

A comparison between the data in Tables 3 and 7 indicates that the minimum vertical curve lengths for the worst-performing driver in a truck with a conventional braking system are always longer than current AASHTO criteria—in some cases by a substantial margin. At the same time, the minimum vertical curve lengths for a truck with an antilock brake system or for the best-performing driver in a truck with a conventional brake system are always shorter than the current AASHTO criteria. Stated another way, both the truck with the antilock brake system and the best-performing driver with a conventional brake system will always have enough stopping sight distance on a vertical curve designed in accordance with AASHTO criteria.

Finally, the data in Table 7 show that the minimum vertical curve lengths are not very sensitive to the difference between 75 and 93 in of driver eye height. The maximum difference in vertical curve lengths between these minimum and average driver eye heights is 600 ft in one extreme case, although most of the differences are substantially shorter.

TABLE 7 MINIMUM VERTICAL CURVE LENGTHS (IN FEET) TO PROVIDE STOPPING SIGHT DISTANCE FOR PASSENGER CARS AND TRUCKS

Algebraic Difference in Percent Grade	Design Speed (mph) 20	30	40	50	60	70
TRUCK (driver eye height = 75 in)						
Conventional Brake System with Worst Performance Driver						
2	60	90	240	420	910	1,570
4	60	180	470	1,020	1,810	3,140
6	70	250	710	1,520	2,720	4,710
8	90	350	940	2,030	3,630	6,280
10	100	430	1,180	2,530	4,530	7,850
Conventional Brake System with Best Performance Driver						
2	60	90	130	260	340	750
4	60	100	210	520	910	1,530
6	60	110	380	780	1,360	2,300
8	60	200	450	1,040	1,810	3,050
10	80	250	630	1,300	2,261	3,820
Antilock Brake System						
2	60	90	120	200	350	510
4	60	90	130	400	700	1,150
6	60	120	300	600	1,040	1,720
8	60	140	400	800	1,400	2,300
10	60	200	500	1,000	1,730	2,870
TRUCK (driver eye height = 93 in)						
Conventional Brake System with Worst Performance Driver						
2	60	90	200	230	720	1,330
4	60	150	380	860	1,530	2,650
6	60	190	600	1,290	2,300	3,980
8	80	290	800	1,710	3,100	5,300
10	100	360	990	2,140	3,820	6,630
Conventional Brake System with Best Performance Driver						
2	60	90	120	220	390	560
4	60	90	220	430	770	1,290
6	60	130	320	660	1,150	1,930
8	60	150	430	880	1,530	2,580
10	60	210	540	1,080	1,910	3,220
Antilock Brake System						
2	60	90	120	190	320	390
4	60	90	190	340	640	1,060
6	60	110	260	560	960	1,590
8	60	120	370	740	1,270	2,120
10	60	180	460	920	1,590	2,650

Note: Based on stopping sight distances shown in Table 6.

Horizontal Sight Obstructions

The differences in stopping sight distance between passenger cars and trucks shown in Table 6 are generally not mitigated by increased driver eye height, as in the case of vertical sight restrictions. As shown in Equation 6, the sight distance requirements for horizontal curves should actually be somewhat higher—as a function of curve radius and superelevation—than for tangents.

CONCLUSIONS

A truck with a conventional brake system driven by a worst-performing driver requires up to 425 ft more stopping sight distance at 70 mph and requires longer crest vertical curves than current AASHTO policy recommends. Specific calculated values of stopping sight distance and crest vertical curve length are given in Tables 6 and 7.

In contrast, a truck with a conventional brake system and driven by a best-performing driver requires only slightly more stopping sight distance than current AASHTO policy and, because of increased driver eye height, requires shorter crest vertical curves than AASHTO recommends. This finding points to the critical role played by driver training and experience in emergency braking maneuvers. Unfortunately, current data do not provide any reliable estimates of the distribution of driver performance in the range between the extremes.

In the worst-case scenario, there may be a need to increase stopping sight distance requirements to accommodate trucks with conventional brake systems. The safety benefits of such a change are not known, however, and it has not been established whether revision of current design criteria would be cost-effective. Changes in current design criteria are not recommended unless a cost-effectiveness analysis indicates that longer vertical curves would produce safety benefits commensurate with the added construction cost.

Trucks with antilock brake systems require less stopping sight distance and significantly shorter crest vertical curves than current AASHTO policy recommends. It appears that trucks with antilock brake systems can stop in the same or less distance than a passenger car. Thus, future government policy and industry practice concerning the use of antilock brake systems have major implications for highway design policy, because it is likely that no changes in current stopping sight distance design policies would be needed to accommodate trucks if antilock brakes were required or widely used.

ACKNOWLEDGMENT

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The findings and conclusions of the paper are those of the authors and do not necessarily represent the views of the Federal Highway Administration.

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Intersection Sight Distance Requirements for Large Trucks

JOHN M. MASON, JR., KAY FITZPATRICK, AND DOUGLAS W. HARWOOD

An analysis has been conducted to determine the sight distance requirements of large trucks at intersections. AASHTO policy is briefly reviewed and related vehicle characteristics are identified. Truck characteristics are updated based on permitted 1982 Surface Transportation Assistance Act design vehicles and published truck acceleration models. The results of sensitivity analyses are compared with current policy and are summarized for each of the intersection sight distance cases considered by AASHTO. The findings imply that current intersection sight distance criteria may not be adequate for trucks when the current AASHTO models are exercised for the representative truck characteristics. Nevertheless, the findings, particularly for Case III intersection sight distance, result in impractically long sight distance requirements. Therefore, the development of alternative approaches for establishing realistic sight distance values is advocated. In particular, a truck driver gap-acceptance concept is proposed for further study. The gap lengths that truck drivers safely accept would be determined through field studies, and sight distance criteria would then be established to ensure that truck drivers on a side road approach would have sight distance at least equal to acceptable gap length.

The 1984 AASHTO Green Book (*1*) classifies intersection sight distance as adequate when a driver has an unobstructed view of the entire intersection and sufficient lengths of the intersecting highway to avoid collisions. The AASHTO policy makes various assumptions of physical conditions and driver behavior, including vehicle speed, vehicle performance capabilities, and distances traveled during perception-reaction time and locked-wheel braking.

The current intersection sight distance policy is based primarily on consideration of the passenger car as the design vehicle. Highway design and operational criteria, however, should consider the characteristics of all vehicles using a facility with reasonable frequency. To address the need for additional information an analysis has been conducted to determine the sight distance requirements of large trucks at intersections. This paper focuses on the types of trucks that have been permitted since the 1982 Surface Transportation Assistance Act (STAA) but are excluded from the AASHTO Green Book. The analysis is a simple extension of the existing AASHTO intersection sight distance models to reflect the characteristics and performance of trucks as well as passenger cars. No specific changes in the AASHTO intersection sight distance policies are suggested on the basis of this analysis. Instead, the results of extending these models to trucks point to deficiencies in the AASHTO models themselves and the need for further research to develop new concepts for use in determining intersection sight distance policy.

J. M. Mason and K. Fitzpatrick, Pennsylvania Transportation Institute, Pennsylvania State University, University Park, Pa. 16802. D. W. Harwood, Midwest Research Institute, Kansas City, Mo. 64110.

INTERSECTION SIGHT DISTANCE POLICIES

The sight distance to be provided at intersections is determined by calculating the unobstructed sight distance for vehicles approaching simultaneously on two crossing roadways or for vehicles accelerating from a stop at an intersection approach. Figure 1 illustrates the current design considerations for these two general situations. The simultaneous approach of vehicles on intersecting approaches is considered at "uncontrolled" intersections or where the minor approach has a posted Yield sign. The consideration of acceleration from a stop assumes that a Stop sign is present on the minor roadway or traffic signalization is provided for all approaches.

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

Case I. No control, but allowing vehicles to adjust speed.
Case II. Yield control where vehicles on the minor intersecting roadway must yield to vehicles on the major intersecting roadway.
Case III. Stop control where traffic on the minor roadway must stop prior to entering the major roadway.
Case IV. Signal control where all legs of the intersecting roadways are required to stop by a Stop sign, or where the intersection is controlled by traffic signals.

Case I—No Control

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

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

$$ISD = 1.47 * V * t \qquad (1)$$

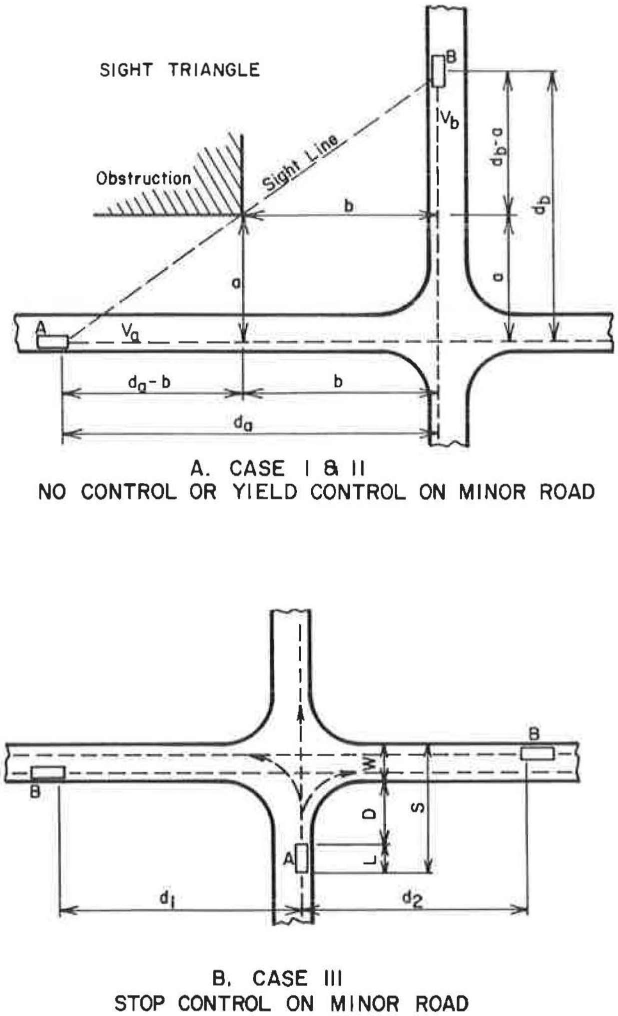

FIGURE 1 Design considerations for intersection sight distance (*1*).

where

ISD = d_a or d_b, minimum intersection sight distance (ft) (see Figure 1A),
V = speed of vehicle (mph),
t = $t_{pr} + t_r$ (sec) (assumed: t = 3.0 sec),
t_{pr} = perception-reaction time (sec) (assumed: t_{pr} = 2.0 sec), and
t_r = time required to regulate speed (sec) (assumed: t_r = 1.0 sec).

An earlier analysis of Case I intersection sight distance by McGee et al. (*2*) focused on its sensitivity to changes in the time needed to regulate speed (assumed by AASHTO as 1 sec). Since deceleration, the vehicle characteristic, is inherent in the 1 sec, a change in the time needed to regulate speed was used as a surrogate for a change in the deceleration rate. Modifying t by ½ sec results in a 17 percent change in the required sight distance. When using this method of testing changes in deceleration rate, it is important to remember that change in the time to regulate speed can represent three different things: a change in the final speed reached, a change in the distance traveled while decelerating, or a change in the deceleration rate. Since the current design standard does not include an explicit term incorporating vehicle deceleration characteristics, the determination of the standard's sensitivity to this characteristic is limited. Because of these limitations, a new formula that incorporates consideration of deceleration rate (d) was proposed by McGee:

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

where

ISD_A = d_a, minimum intersection sight distance for Vehicle A (ft) (see Figure 1A),
V_A = design speed for Vehicle A (mph),
t_{pr} = perception-reaction time (sec) (assumed: t_{pr} = 2.0 sec),
W = width of roadway on which Vehicle A is traveling (ft),
V_B = design speed of Vehicle B (mph), and
d_A = deceleration rate of Vehicle A (mph/sec) (note that if the vehicle accelerates, d_A has a negative value).

Equation 2 explicitly considers deceleration rate, but it does not incorporate vehicle length and is highly dependent on perception-reaction time. Vehicle length consideration is presented later in the sensitivity analysis section of this paper.

Case II—Yield Control

The sight distance for the vehicle operator on the minor road must be sufficient to allow him to observe a vehicle on the major roadway approaching from either the left or the right and to bring the vehicle to a stop before he reaches the intersecting roadway. This maneuver requires sight distance equal to stopping sight distance, which is a function of perception-reaction time and braking time.

Case III — Stop Control

The AASHTO Green Book states: "Where traffic on the minor road of an intersection is controlled by Stop signs, the driver of the vehicle on the minor road must have sufficient sight distance for a safe departure from the stopped position, even though the approaching vehicle comes in view as the stopped vehicle begins its departure movements" (Figure 1B). Three basic maneuvers occur at the average intersection:

1. Traveling across the intersecting roadway by clearing traffic on both the left and the right of the crossing vehicle,
2. Turning left into the crossing roadway by first clearing traffic on the left and then entering the traffic stream with vehicles from the right, and
3. Turning right into the intersecting roadway by entering the traffic stream with vehicles from the left.

Consequently, there are three separate sight distance criteria for a vehicle stopped at an intersection. (These conditions are referred to as Cases A, B, and C in Figure IX-23 of the AASHTO Green Book.)

Case III-A—Crossing Maneuver

As stated in the AASHTO Green Book "the sight distance for a crossing maneuver is based on the time it takes for the stopped vehicle to clear the intersection and the distance that a vehicle will travel along the major road at its design speed in that amount of time." Case B in Figure 1 illustrates this condition. The sight distance may be calculated from the following equation:

$$ISD = 1.47 * V * (J + t_a) \tag{3}$$

where

- ISD = d_1 or d_2, sight distance along the major highway from the intersection (ft),
- V = design speed on the major highway (mph),
- J = sum of the perception time and the time required to actuate the clutch or an automatic shift (sec) (assumed: J = 2.0 sec),
- t_a = time required to accelerate and traverse the distance (S) to clear the major highway pavement (sec) (values of t_a can be read directly from AASHTO Figure IX-21 for nearly level conditions for a given distance S),
- S = $D + W + L$, the distance that the crossing vehicle must travel to clear the major highway (ft) (see Figure 1B),
- D = distance from the near edge of pavement to the front of a stopped vehicle (ft) (assumed: D = 10 ft),
- W = pavement width along path of crossing vehicle (ft), and
- L = overall length of vehicle (ft) (AASHTO Green Book values are 19, 30, 50, 55, and 65 ft for the P, SU, WB-40, WB-50, and WB-60 vehicles, respectively).

McGee et al. (*2*) found Case III-A to be generally insensitive to changes in vehicle characteristic values used in current AASHTO criteria. The current criteria are based on a truck with a length of 55 ft. Increasing the truck length to between 60 and 70 ft increased the required intersection sight distance by approximately 10 percent. An important concern noted by McGee et al. (*2*) is that the AASHTO curves for t_a (time to accelerate) were established from empirical data observed prior to 1954.

Case III-B—Turning Left into a Crossroad

A vehicle turning left into a crossroad should have, as a minimum, sight distance to a vehicle approaching from the right at the design speed. The turning vehicle should be able to accelerate to the average running speed by the time the approaching vehicle gets within a certain tailgate distance after reducing its speed to the average running speed, or the turning vehicle should be able to accelerate to the design speed by the time the approaching vehicle gets within a certain tailgate distance while maintaining the design speed. Figure IX-24 in the AASHTO Green Book describes the details of this case.

AASHTO states that the required sight distances for trucks turning left onto a crossroad will be substantially longer than those for passenger cars. AASHTO further indicates that the sight distance for trucks can be determined using appropriate assumptions for vehicle acceleration rates and turning paths. The specific assumptions, however, are not detailed in AASHTO policy. As presented, the case for this standard lacks sufficient information to derive the design curves for determining required sight distance dimensions.

Case III-C—Turning Right into a Crossroad

A right-turning vehicle must have sufficient sight distance to vehicles approaching from the left to complete its right turn and to accelerate to the running speed before being overtaken by traffic approaching from the left and traveling at the same running speed. The Case III-C policy is described in Figure IX-25 in the AASHTO Green Book. The sight distance requirement for a right-turn maneuver is only a few feet less than that required for a left-turn maneuver. As in Case III-B, AASHTO indicates that sight distances for trucks need to be considerably longer than for passenger vehicles, but sufficient information is lacking to derive the design curves for determining required sight distance dimensions.

Case IV—Signal Control

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

SENSITIVITY ANALYSIS

Table 1 contains a summary of the intersection sight distance parameters used in the AASHTO Green Book and the values of the vehicle-related parameters that were varied in the subsequent sensitivity analyses. The values in the AASHTO column are those used in the current criteria. They include driver characteristics (perception-reaction time) and vehicle characteristics (deceleration or acceleration time, stopping distance, and vehicle length). "Modifications for Truck Characteristics" in Table 1 represent updated truck characteristics data, including truck lengths, based on permitted STAA design vehicles and stopping sight distances for trucks. The sources of the truck characteristics data are documented below. The application of these data to derive sight distances for trucks for each intersection case is presented in the following sections.

The revised passenger car and truck acceleration rates for Case I are based on the work of McGee et al. (*2*). The stopping

TABLE 1 SUMMARY OF TRUCK CHARACTERISTICS FOR INTERSECTION SIGHT DISTANCES (ISD)

	AASHTO (1)			Modifications for Truck Characteristics	
Case	Perception-Reaction Time (sec)	Deceleration/ Acceleration Time or Distance	Length of Vehicle	Deceleration/ Acceleration Time or Distance	Length of Vehicle
CASE I ISD = 1.47Vt $t = t_{pr} + t_r$	$t_{pr} = 2.0$	$t_r = 1.0$ seconds	NA	New Equation $d_A = 5.5$ mphps for pc $d_A = 3.63$ mphps for tractor-trailer combinations	70' tractor semi-trailer truck 75' tractor semi-trailer-full trailer truck (double bottom)
CASE II ISD = SSD = $1.47Vt_{pr} + \frac{V^2}{30(f+g)}$	$t_{pr} = 2.5$	AASHTO's SSD Values Speed SSD 20 mph 125' 30 mph 200' 40 mph 325' 50 mph 475' 60 mph 650' 70 mph 850'	NA	SSD for truck with worst and best performing drivers Driver Performance Speed Worst Best 20 mph 150' 125' 30 mph 300' 250' 40 mph 500' 375' 50 mph 725' 525' 60 mph 975' 700' 70 mph 1275' 900'	NA
CASE III-A ISD = $1.47V(J+t_a)$ t_a = function(D+W+L)	J = 2.0	t_a from AASHTO Figure IX-21	19' PC 30' SU 55' WB-50	t_c from Gillespie's equation (5)	70' tractor semi-trailer truck 75' tractor semi-trailer-full trailer truck (double bottom)
CASE III-B & III-C	From AASHTO Figure IX-27			See Table 5	70' tractor semi-trailer truck 75' tractor semi-trailer-full trailer truck (double bottom)

sight distances used for Case II are those derived by Harwood, Glauz, and Mason (in this Record), based on estimates of truck braking distances developed by Fancher (*3*). These distances represent controlled braking by an empty truck on a poor, wet road with relatively good radial tires (at least 12/32 in of tread depth). The truck braking performance of drivers varies widely as a result of driver experience and expertise: many truck drivers lack experience in emergency braking, and different drivers accept varying amounts of "risk" in what is potentially a hazardous operation that could lead to truck jackknifing. Fancher (*3*) found that the worst-performing driver has a braking efficiency of approximately 62 percent of the vehicle capability, while the best-performing drivers can achieve nearly 100 percent of vehicle capability. A range of stopping sight distances appropriate for both the worst and best drivers (62 to 100 percent driver control efficiency) is considered in this paper.

Clearance times for trucks crossing intersections in Case III-A are based on a relationship developed by Gillespie (*4*), presented later in this paper. Truck acceleration performance for Cases III-B and III-C is based on test track data collected by Hutton (*5*).

Truck lengths of 70 and 75 ft were considered. An overall length of 70 ft represents a STAA tractor semitrailer truck with a 53-ft trailer unit. The overall length of 75 ft represents a STAA "double bottom" truck with a conventional cab-behind-engine tractor and two 28-ft trailers.

Case I—No Control

The current formula for Case I intersection sight distance includes the driver's perception-reaction time. The AASHTO formula implicitly accounts for vehicle characteristics through the 1.0 sec time to regulate speed assumption.

As discussed earlier, McGee et al. (*2*) proposed an alternative equation for Case I intersection sight distance that explicitly included deceleration rate (see Equation 2). The McGee equation estimates sight distances that are less than the AASHTO criteria. The equation does not adequately address Case I intersection sight distance because it does not consider vehicle lengths. A tractor-trailer requires more time to cross an intersection than a passenger car because of its increased length. Therefore, a further modification of the equation is proposed to account for the length of the crossing vehicle (B) and the deceleration rate of the conflicting vehicle (A):

$$ISD_A = 1.47 V_A t_{pr} + (W + L_B)\frac{V_A}{V_B} - \frac{d_A (W + L_B)^2}{2.93 V_B^2} \qquad (4)$$

where

ISD_A = d_a, minimum intersection sight distance for Vehicle A (ft) (see Figure 1A),
V_A = design speed for Vehicle A (mph),
V_B = design speed for Vehicle B (mph),
t_{pr} = perception-reaction time (sec) (assumed: t_{pr} = 2.0 sec),
W = width of roadway on which Vehicle A is traveling (ft),
L_B = vehicle length of Vehicle B (ft), and
d_A = deceleration rate of Vehicle A (mph/sec) (if the vehicle accelerates, d_A is a negative value).

Table 2 and Figure 2 compare the Case I intersection sight distances based on the AASHTO Green Book criteria and those given by Equation 4 for truck lengths of 70 and 75 ft. The results indicate that longer trucks require more distance than is provided by the AASHTO criteria for Vehicle B speeds up to 60 mph. The percent change in the sight distance required for Vehicle A ranges from an increase of 69 percent (when V_A = 70 mph and V_B = 20 mph) to a decrease of 5 percent (when V_A = 20 mph and V_B = 70 mph).

Use of Equation 4 for Case I intersection sight distance is recommended because it explicitly considers both deceleration rate and vehicle length. Sight distances calculated from this formula are more sensitive to the vehicle length than to the deceleration term. The revised equation is still highly dependent on the driver perception-reaction time.

Case II—Yield Control

The Case II intersection sight distance procedure is merely an application of the AASHTO stopping sight distance formula, using the revised stopping sight distances for trucks shown in Table 1. The percent increase in sight distance for the worst- and best-performing truck drivers in comparison with the current AASHTO criteria is shown in Table 3.

Case III-A—Crossing Maneuver

The current AASHTO criteria for Case III-A intersection sight distance include two vehicle characteristics: vehicle acceleration from a stop and vehicle length. Both characteristics are used to determine the acceleration time parameter (t_a) used in the criteria. AASHTO Green Book Figure IX-21 provides distance versus time curves for acceleration by a passenger car, a single-unit truck, and a WB-50 truck. Vehicle length is necessary to establish the length of the hazard zone, in addition to the distance from the front of the vehicle to the edge of the intersecting pavement (AASHTO assumes 10 ft) and the width of the intersection. Table 4 lists the sight distance required for an AASHTO WB-50 truck to cross a 30-ft intersection, based on the AASHTO acceleration performance curve (AASHTO Green Book Figure IX-21).

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

The acceleration time to clear a hazard zone has also been calculated using an equation developed by Gillespie (*4*). The

TABLE 2 COMPARISON OF CASE I INTERSECTION SIGHT DISTANCES (ISD)

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

See Figure 1-A for vehicle A and vehicle B sight triangles.

ASSUMPTIONS: W = 24 ft

d_A for 70' tractor semi-trailer truck = 3.63 mphps

d_A for 75' tractor semi-trailer-full trailer truck = 3.63 mphps

d_A values from Table 33 in reference 3, 85 percentile average deceleration rate on wet pavement with an initial speed of 40 mph

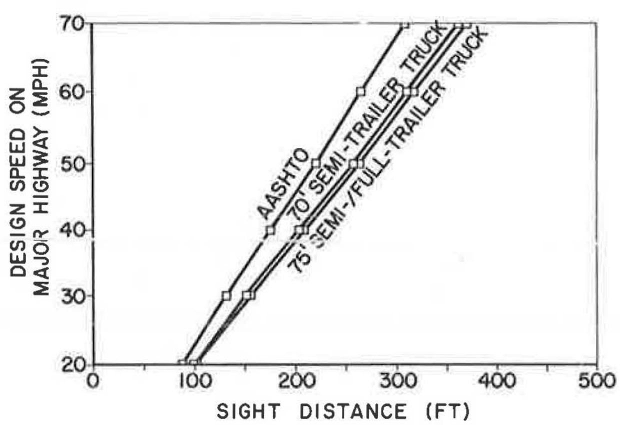

FIGURE 2 Comparison of Case I intersection sight distances (Vehicle B speed = 40 mph).

time (t_c) required for a truck to clear a hazard zone—starting from a full stop and remaining in initial gear during the maneuver—can be estimated by the following equation:

$$t_c = 0.682 \frac{L_{HZ} + L_T}{v_{mg}} + 3.0 \text{ (sec)} \quad (5)$$

where

L_{HZ} = length of the hazard zone (ft),
L_T = length of the truck (ft), and
v_{mg} = maximum speed in a selected gear (mph) (determined by Gillespie as 8 mph for a level surface).

Gillespie also presented a maximum speed in initial gear versus grade curve for determination of clearance time for trucks accelerating on a grade. Equation 5 assumes that the gear design, engine speed, and the tire size are such that the truck's

TABLE 3 PERCENT INCREASE IN INTERSECTION SIGHT DISTANCES (ISD) OVER AASHTO CRITERIA

Speed	AASHTO SSD (1) (ft)	Worst-Performance Truck Driver ISD (ft)	Worst-Performance Truck Driver Percent Increase	Best-Performance Truck Driver ISD (ft)	Best-Performance Truck Driver Percent Increase
20	125	150	20.00	125	0.00
30	200	300	50.00	250	25.00
40	325	500	53.85	375	15.38
50	475	725	52.63	525	10.53
60	650	975	50.00	700	7.69
70	850	1275	50.00	900	5.88

TABLE 4 COMPARISON OF CASE III-A INTERSECTION SIGHT DISTANCES (ISD)

	AASHTO	GILLESPIE--INTERSECTION CLEARANCE TIME	
VEHICLE B SPEED (mph)	WB-50 TRUCK (55') ISD (ft)	70' TRACTOR SEMI-TRAILER TRUCK ISD (ft)	75' TRACTOR SEMI-TRAILER-FULL-TRAILER TRUCK ISD (ft)
20	370	423	435
25	463	528	544
30	556	634	653
35	648	740	762
40	741	845	870
45	833	951	979
50	926	1057	1088
55	1019	1162	1197
60	1111	1268	1306
65	1204	1374	1414
70	1297	1479	1523

Assumed:
Width of pavement: 30'
Distance from edge of pavement to front of vehicle: 10'
t_a determined from Figure IX-21 in AASHTO Green Book
t_a = 10.6 seconds for 55' truck
t_c determined from Gillespie's equation
t_c = 12.38 seconds for 70' truck
t_c = 12.80 seconds for 75' truck

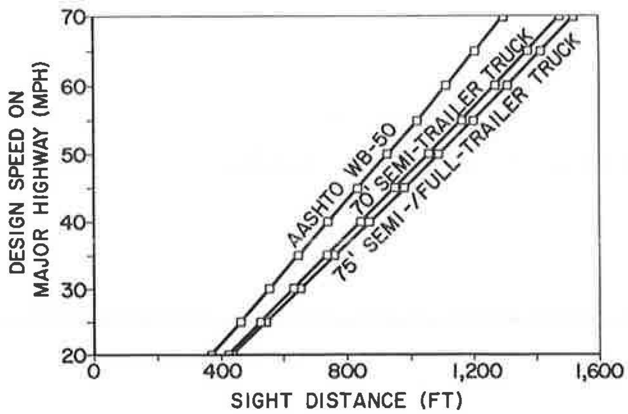

FIGURE 3 Comparison of Case III-A intersection sight distances.

maximum speed is 60 mph. It also assumes that a truck will remain in the initial gear without shifting while negotiating the hazard zone.

Intersection sight distances based on the Gillespie model are also shown in Table 4. Use of the Gillespie model for a 70- and 75-ft truck resulted in a 17 and 21 percent increase in time, respectively, compared with an AASHTO WB-50 truck. These longer times produce a 14 percent increase in sight distance for a 70-ft tractor semitrailer truck and a 17.5 percent increase for a 75-ft tractor semitrailer–full-trailer truck. Figure 3 illustrates the results presented in Table 4.

Cases III-B and III-C—Turning Left or Right into a Crossroad

The vehicle characteristics considered in intersection sight distance for Cases III-B and III-C are acceleration rate, vehicle length, and vehicle turning path. The acceleration rate is a function of the distance traveled and the vehicle type. Cases III-B and III-C require considerably longer sight distances than Case III-A because more time is needed to turn left or right and accelerate to the design speed than is required to cross the intersecting roadway.

Because the intersection sight distance criteria presented in the Green Book for Cases III-B and III-C lack the information to determine the parameter values needed to derive the design curves of AASHTO's Figure IX-27, the following assumptions were necessary:

1. Vehicle B (vehicle on major highway) maintains design speed throughout the turning maneuver by Vehicle A.
2. In 1970, Hutton published a paper on the acceleration performance of highway diesel trucks accelerating from a stopped position to a maximum speed on a straight and level surface (*5*). The acceleration distance and time for Vehicle A are based on the Hutton curve data. The distance traveled by Vehicle A during a turning maneuver can be estimated for trucks with weight-to-horsepower ratios of 100, 200, and 300. Hutton also estimates the time (t_t) for Vehicle A to complete the turning maneuver. Table 5 gives the distance and time that would be required for Vehicle A to accelerate from a stop to various speeds.
3. The distance traveled by Vehicle B is equal to the design speed of the major highway multiplied by the time for Vehicle A to accelerate from a stopped position to the design speed.
4. A proposed methodology for quantifying AASHTO's tailgate distance is to consider the variation in average spacing between vehicles traveling at selected design speeds. This dimension is referred to as the "minimum separation" between the front bumper of the vehicle on the major road and the rear bumper of the turning vehicle (see Figure 4). No field data are available on the minimum separations actually accepted by drivers in making turning maneuvers at intersections. One approach to determine the minimum separation is to determine the space gaps that drivers use when traveling at short headways. In other words,

$$MS = 1.47h_{min} V - L \tag{6}$$

where

MS = minimum separation (ft),
h_{min} = minimum acceptable headway (sec),
V = vehicle speed (mph), and
L = vehicle length (ft).

The following shows the minimum separations derived from this approach for a minimum acceptable headway of 1 sec and a vehicle length of 19 ft:

Design Speed (mph)	*Minimum Separation (ft)*
25	18
30	25
35	32
40	40
45	47
50	55
55	62
60	69
65	77
70	84

It is known that some vehicles will travel at these minimum separations. It is possible that even shorter minimum separations might be maintained for brief intervals during a turning maneuver.

Using the above assumptions and the information presented in the AASHTO Green Book, the following equations were used in this sensitivity analysis (see Figure 4 for dimensions in Case III-B):

$$ISD_A = Q - H \tag{7}$$

$$Q = 1.47 * V_B * t_A \tag{8}$$

$$t_A = t_t + J \tag{9}$$

$$H = P - D_{np} - MS - L + R \tag{10}$$

$$D_{np} = \pi * R/2 \tag{11}$$

where

ISD_A = d_1 or d_2, sight distance along the major highway from the intersection for Vehicle A (ft) (see Figure 1B),
Q = distance traveled by Vehicle B during Vehicle A's turning maneuver (ft),
H = distance of Vehicle B from intersection when at

TABLE 5 ACCELERATION TIMES AND DISTANCES FOR TRUCKS (*6*)

	WEIGHT/HORSEPOWER RATIO					
	100 LB/HP		200 LB/HP		300 LB/HP	
Speed (mph)	Time (sec)	Distance (ft)	Time (sec)	Distance (ft)	Time (sec)	Distance (ft)
25	15	400	20	500	25	600
30	17	500	25	700	37	1000
35	24	800	35	1100	50	1600
40	30	1100	49	1800	65	2400
45	38	1600	60	2500	85	3700
50	47	2200	76	3600	107	5500
55	57	3000	92	4900	*	*
60	67	4000	107	6400	*	*
65	77	5000	*	*	*	*
70	85	6000	*	*	*	*

*Information not available

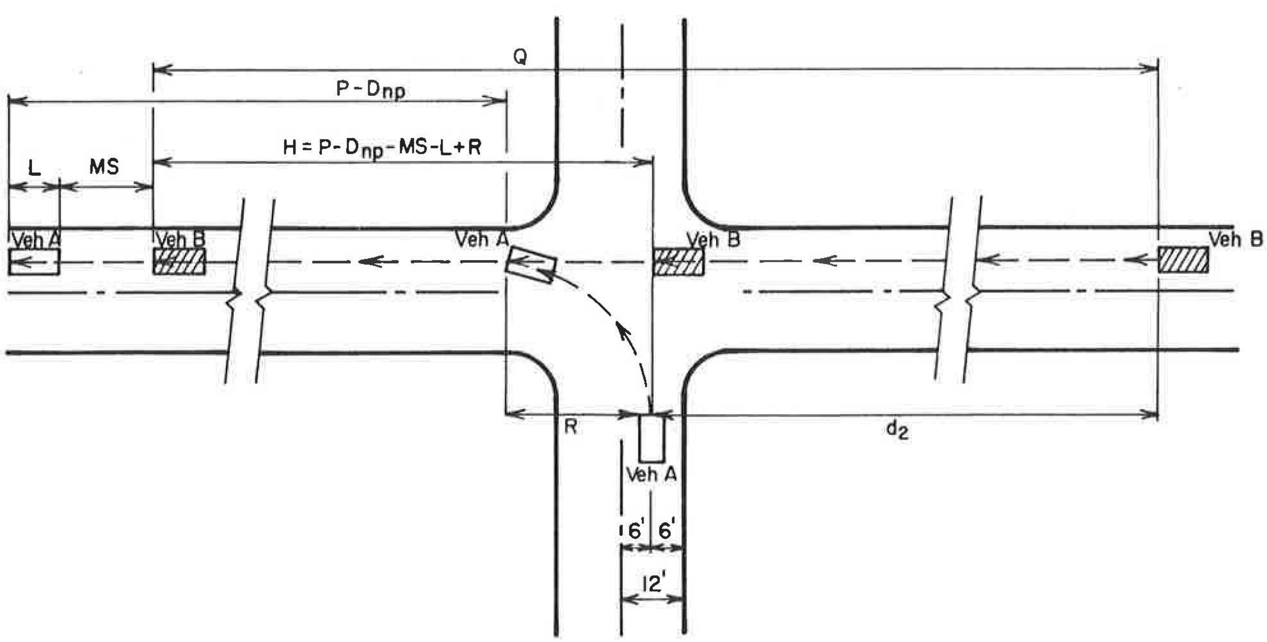

FIGURE 4 Distances considered in Case III-B criteria.

assumed minimum separation distance from Vehicle A (ft),

V_B = velocity of Vehicle B (mph),

t_A = time for a stopped vehicle to move into traffic stream and accelerate to design speed (sec),

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

t_t = time for Vehicle A to complete the turning maneuver (sec) (based on Hutton data),

P = total distance traveled by Vehicle A from stopped position to location when design speed is achieved (ft) (based on Hutton data),

D_{np} = distance Vehicle A traveled during the turning maneuver that is not parallel to highway (ft),

MS = minimum separation (ft),

L = length of Vehicle A (ft),

R = radius of turn for Vehicle A (ft) (based on assumed values from Table IX-20 in the AASHTO Green Book).

Any differences in sight distance lengths between Case III-B and Case III-C would be caused by the different turning radii (R) between a left turn and a right turn.

The percent changes in required sight distance resulting from changes in the vehicle characteristics were determined by comparing the sight distances calculated from the above assumptions (for trucks with 200 and 300 weight-to-horsepower ratio) with the sight distances shown in AASHTO Figure IX-27, Curve *B-2a* and *Ca*. Table 6 presents the sight distances calculated with the above assumptions and their percent differences from the AASHTO criteria. Figure 5 compares the revised sight distances in Table 6 directly with the AASHTO sight distances. For each weight-to-horsepower ratio, the revised intersection sight distances are greater (between 51 and 139 percent) than the AASHTO criteria.

SUMMARY OF FINDINGS

A revised model developed in this study indicates that Case I intersection sight distance is quite sensitive to vehicle length, which is not considered in the current AASHTO criteria. Sensitivity analysis results indicate that trucks require

TABLE 6 COMPARISON OF CASE III-B AND III-C INTERSECTION SIGHT DISTANCES (ISD)

SPEED VEH B V_B (mph)	DISTANCE VEH A P (ft)	TIME VEH A t_t (sec)	DISTANCE VEH B Q (ft)	MINIMUM SEPARATION MS (ft)	CALCULATED ISD ISD_A (ft)	AASHTO ISD (ft)	PERCENT INCREASE
design vehicle - 200 lbs/hp, 70-foot							
25	500	20	809	18	430	325	32
30	700	25	1,191	25	620	450	38
35	1,100	35	1,904	32	940	580	62
40	1,800	49	2,999	40	1,343	750	79
45	2,500	60	4,101	47	1,753	950	84
50	3,600	76	5,733	55	2,292	1,190	93
55	4,900	92	7,600	62	2,866	1,440	99
60	6,400	107	9,614	69	3,387	1,730	96
65-70	DATA UNAVAILABLE						
design vehicle - 300 lbs/hp, 75-foot							
25	600	25	992	18	519	325	60
30	1,000	37	1,720	25	854	450	90
35	1,600	50	2,675	32	1,217	580	110
40	2,400	65	3,940	40	1,689	750	125
45	3,700	85	5,755	47	2,211	950	133
50	5,500	107	8,012	55	2,675	1,190	125
55-70	DATA UNAVAILABLE						

NOTE: Radius of turn, R = 60 feet

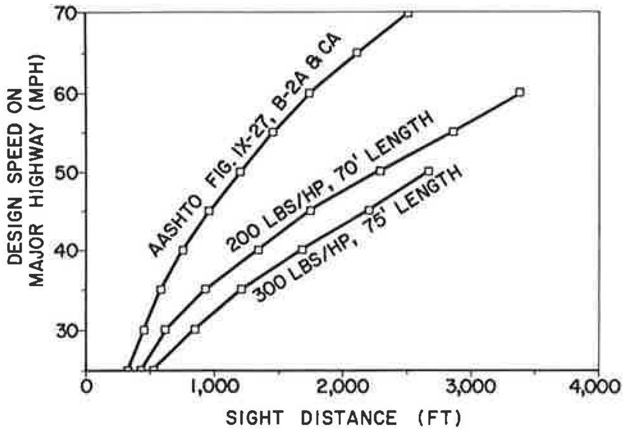

FIGURE 5 Comparison of Case III-B and III-C intersection sight distances.

greater Case I intersection sight distance than the current AASHTO criteria for all approach speeds considered and for all crossing vehicle speeds up to 60 mph.

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

A sensitivity analysis found that 70- and 75-ft combination trucks require substantially longer intersection sight distance than an AASHTO WB-50 truck for Case III-A. In particular, intersection clearance times based on the model developed by Gillespie indicate that a 70-ft truck requires 14 percent more sight distance than an AASHTO WB-50 truck and that a 75-ft truck requires 17.5 percent more sight distance.

The sensitivity analysis also found that the selected trucks would require substantially more intersection sight distance than passenger cars for Cases III-B and III-C. The additional sight distance requirements of trucks vary as a function of weight-to-horsepower ratio. A 200-lb/hp, 70-ft truck requires between 51 and 103 percent additional sight distance compared with a passenger car, and a 300-lb/hp, 75-ft truck requires between 78 and 139 percent additional sight distance.

The analysis presented in this paper is based on extending current AASHTO intersection sight distance models with data on the characteristics and performance of vehicles permitted since the 1982 STAA, but excluded from the AASHTO Green Book. Each intersection case resulted in increased sight distance requirements. For Cases I, II, and III-A, the largest additional truck sight distance requirements range from approximately 125 ft to 450 ft. Cases III-B and III-C result in an increase in sight distances of nearly 1,700 ft in some situations. The existing criteria for Cases III-B and III-C can require intersection sight distances of up to 1,700 ft. The revised requirements for trucks can be as large as 3,400 ft.

It is clear from operational experience that sight distances as long as 3,400 ft are not necessary for safe operations at intersections, even where large trucks are present. Very few intersections have such long sight distances available, and it is unlikely that either passenger car or truck drivers could accurately judge the location and speed of an oncoming vehicle at a distance of 3,400 ft. Rather, this result indicates that the current AASHTO model for Cases III-B and III-C for truck intersection sight distance, on which this analysis is based, is unrealistic. In particular, it is unrealistic to assume that potentially conflicting vehicles on the main road will make only minor adjustments in speed if a truck from the side road makes a left or right turn.

There is a need to revise or replace the AASHTO model for Cases III-B and III-C intersection sight distance, especially for trucks. Two alternative approaches are available. First, the AASHTO model could be revised to incorporate deceleration by the main road vehicle when a truck executes a turning maneuver from the side road. Although more realistic, this approach would increase the complexity of the model. The deceleration behavior of drivers would have to be based on field studies for a range of vehicle types, driver types, intersection geometrics, and approach speeds.

An alternative approach to establishing practical sight distance values is to base the criteria on gap lengths safely accepted by the side road trucks. The sight distance criteria should be developed to ensure that truck drivers on the side road would have sight distance that is at least equal to their acceptable gap length. Sight distances established from gap acceptance investigations would better represent actual operations at an intersection.

Truck drivers need to view an adequate length of roadway to determine if there is an adequate gap on the major road to safely complete the maneuver. The gap lengths that truck drivers accept can be estimated through field studies. Factors that should be considered in the studies include

- Location of intersection (rural or urban),
- Traffic volume (peak hour, daily, and seasonal variations),
- Vehicle mix characteristics (composition and vehicle configuration), and
- Geometric elements (horizontal and vertical alignment and cross-section descriptions).

FUTURE RESEARCH

Field studies can provide the data to further develop the revised concepts for Case III truck intersection sight distance. The specific results that are necessary to evaluate the potential of a gap acceptance concept for intersection sight distance include

1. Gap distances the trucks on minor roads will accept and reject during their maneuver onto or across the major roadway,
2. Development of a speed profile (deceleration behavior) for major road vehicles during the maneuvers of a truck on a minor road,
3. Acceleration characteristics (time/distance relationships) of the truck on the minor roadway during a crossing or turning maneuver, and
4. Safe minimum separation distance between the turning vehicle (truck) and the oncoming vehicle.

These data could also be used to revise the current AASHTO model to incorporate deceleration by the vehicle on the main road. The preliminary findings presented in this paper indicate that the issues regarding truck intersection sight distance are also applicable to the needs of passenger car drivers. As such, the future research efforts identified above are equally important to consider in examining modifications to AASHTO's current intersection sight distance policy.

ACKNOWLEDGMENT

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Passing Sight Distance Design for Passenger Cars and Trucks

DOUGLAS W. HARWOOD AND JOHN C. GLENNON

Safe and effective passing zones on two-lane highways require both adequate sight distance to opposing vehicles and adequate passing zone length. Current design and marking criteria for passing zones on two-lane highways are reviewed. A recently developed model of the kinematic relationships among the passing, passed, and opposing vehicles is employed to evaluate the current design and marking criteria. The model is used both to evaluate the current criteria, which are based solely on passenger cars, and to consider the passing requirements when the passed vehicle, the passing vehicle, or both, are large trucks.

Two major aspects of passing and no-passing zone marking criteria determine the safety and operational effectiveness of the passing and no-passing zones marked on two-lane highways: passing sight distance and passing zone length. Safe passing maneuvers require both adequate passing sight distance and adequate passing zone length. Recent debate over passing zone design and marking criteria, however, has tended to focus only on passing sight distance and to ignore passing zone length. This paper gives thorough consideration to the important roles of both these factors based on recent advances in modeling the kinematic relationships among the passing, passed, and opposing vehicles.

Current passing and no-passing zone marking criteria use the passenger car as the design vehicle. This paper considers the effect on passing sight distance and passing zone length requirements if the passed vehicle, the passing vehicle, or both are large trucks.

CURRENT PASSING SIGHT DISTANCE CRITERIA

Passing sight distance is needed where passing is permitted on two-lane, two-way highways to ensure that passing drivers, who use the lane normally reserved for opposing traffic, have a sufficiently clear view ahead to minimize the possibility of collision with an opposing vehicle.

Design Criteria

The current design criteria for passing sight distance on two-lane highways in the AASHTO Green Book (*1*) are based on the results of field studies (*2*, *3*) conducted between 1938 and 1941 and validated by another study (*4*) conducted in 1958. The AASHTO policy defines the minimum passing sight distance as the sum of the following four distances:

d_1 = distance traveled by the passing vehicle during perception-reaction time and during initial acceleration to the point of encroachment on the left lane,
d_2 = distance traveled by the passing vehicle while it occupies the left lane,
d_3 = distance between passing vehicle and opposing vehicle at the end of the passing maneuver (that is, clearance distance), and
d_4 = distance traveled by an opposing vehicle for two-thirds of the time the passing vehicle occupies the left lane, or ⅔ of d_2.

Design values for these four distances were derived using the field study results and the following assumptions:

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

The design values for the four components of passing sight distance, shown in Figure III-2 of the AASHTO Green Book, are presented here as Figure 1. Table 1 shows the derivation of the design values for passing sight distance, which is also shown in Figure 1. The columns in Table 1 not headed by a value of design speed represent the field study results from the sources cited earlier (*2*–*4*). The columns headed by design speeds of 20 mph through 70 mph contain values that were interpolated or extrapolated from the field data presented in the intervening columns.

It should be noted that the speeds used to compute the design values for passing sight distance in Table 1 differ from the design speed of the highway. The speed of the passed vehicle is assumed to be equal to the average running speed of traffic (as represented by the intermediate volume curve in

D. W. Harwood, Midwest Research Institute, Kansas City, Mo. 64110. J. C. Glennon, John C. Glennon, Chartered, 8340 Mission Rd., Suite B-12, Prairie Village, Kans. 66206.

TABLE 6 SIGHT DISTANCE REQUIREMENTS FOR PASSING BY TRUCKS

Design or Prevailing Speed (mph)	AASHTO Policy	MUTCD Criteria	Required Passing Sight Distance (ft)	
			Truck Passing Passenger Car	Truck Passing Truck
20	800	-	350	350
30	1,100	500	600	675
40	1,500	600	875	975
50	1,800	800	1,125	1,275
60	2,100	1,000	1,375	1,575
70	2,500	1,200	1,625	1,875

NOTE: Based on revised Glennon Model.

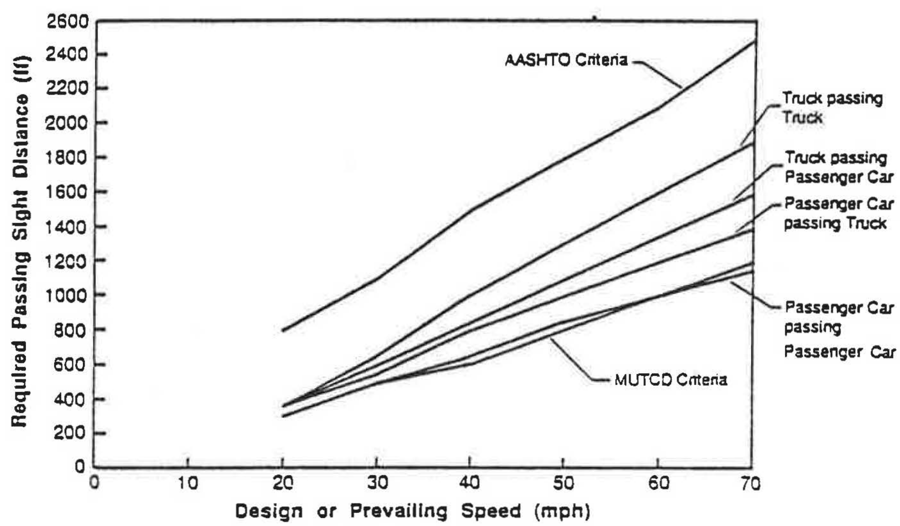

FIGURE 2 Required passing sight distance for passenger cars and trucks in comparison with current criteria.

a deceleration rate of 5 ft/sec^2 (0.15 g), which would be a comfortable deceleration rate on a dry pavement and a critical deceleration rate for a poorly performing driver on a poor, wet pavement, has been assumed.

Table 6 presents the passing sight distance requirements for a 75-ft truck passing a 19-ft passenger car under the assumptions discussed above. The passing sight distance requirements for a truck passing a passenger car are 25 to 425 ft more than for a passenger car passing a passenger car, depending on speed.

Truck Passing Truck

The passing sight distance requirements for a truck passing a truck have also been examined and are presented in Table 6. The analysis was analogous to that done above for a truck passing a passenger car, except that the passed vehicle length was changed to 75 ft. The passing sight distance requirements for a truck passing another truck were found to be 25 to 675 ft longer than for a passenger car passing a passenger car, depending on speed.

Comparison of Results

Figure 2 compares the passing sight distance requirements determined in the sensitivity analysis with the current AASHTO and MUTCD policies. The figure indicates that the current MUTCD criteria are in good agreement with the requirements for a passenger car passing another passenger car. The other passing scenarios—passenger car passing truck, truck passing passenger car, and truck passing truck—each require progressively more sight distance, but substantially less than the current AASHTO criteria.

Effect of Driver Eye Height at Crest Vertical Curves

Where passing sight distance is restricted by a vertical curve, the truck driver has an advantage over a passenger car driver because of greater eye height. As with stopping sight distance, however, the truck driver has no comparable advantage when passing sight distance is restricted by a horizontal sight obstruction, such as a wall or a line of trees on the inside of a horizontal curve.

Table 7 presents the required minimum vertical curve lengths

TABLE 7 MINIMUM VERTICAL CURVE LENGTHS TO MAINTAIN REQUIRED PASSING SIGHT DISTANCE

Algebraic Difference in Grade (%)	Design or Prevailing Speed (mph) 20	30	40	50	60	70
Passenger Car Passing Passenger Car[a]						
2	70	180	320	500	680	940
4	140	360	640	980	1,280	1,630
6	210	540	890	1,240	1,540	1,890
8	280	670	1,020	1,370	1,670	2,020
10	350	750	1,100	1,450	1,750	2,100
Passenger Car Passing Truck[a]						
2	80	220	420	680	1,020	1,360
4	160	430	830	1,280	1,730	2,130
6	240	640	1,090	1,540	1,990	2,390
8	320	770	1,220	1,670	2,120	2,520
10	400	850	1,300	1,750	2,200	2,600
Truck Passing Passenger Car[b]						
2	60	180	370	610	910	1,270
4	120	350	740	1,210	1,710	2,210
6	180	520	1,060	1,560	2,060	2,560
8	240	680	1,230	1,730	2,230	2,730
10	300	790	1,340	1,840	2,340	2,820
Truck Passing Truck[b]						
2	60	220	460	790	1,200	1,690
4	120	440	920	1,510	2,110	2,710
6	180	660	1,260	1,860	2,460	3,060
8	240	830	1,430	2,030	2,630	3,230
10	300	940	1,540	2,140	2,740	3,340

[a] Based on sight distance requirements from Table 5 for passenger car driver eye height of 42 in.

[b] Based on sight distance requirements from Table 6 for truck driver eye height of 75 in.

Note: Curve lengths are expressed in feet.

to maintain passing sight distance over a crest for the four passing scenarios addressed in Tables 5 and 6. Table 7 is based on an eye height of 42 in for a passenger car driver and 75 in for a truck driver. The use of 75 in to represent truck driver eye height is very conservative; the literature shows that truck driver eye height ranges from approximately 71.5 to 112.5 in (*17–19*). Sensitivity analyses for the average truck driver eye height of 93 in did not yield vertical curve lengths much shorter than those for the 75-in eye height.

Table 7 indicates that increased driver eye height partially compensates for the greater sight distance requirements of trucks. For all speeds above 20 mph, a longer minimum vertical curve length is required to maintain adequate passing sight distance for passing maneuvers involving trucks than for a passenger car passing another passenger car. Nevertheless, except at high speeds and when there are large algebraic differences in grades (sharp crests), a truck can safely pass a passenger car on any vertical curve where a passenger car can safely pass a truck.

REVISED CRITERIA FOR PASSING ZONE LENGTH

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

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

- The distance required to complete a pass is the sum of the initial maneuver distance (d_1) and the distance traveled in the left lane (d_2).
- The passing driver does not begin to accelerate in prep-

aration for the passing maneuver until the beginning of the passing zone is reached.

- The initial maneuver distance (d_1) for passes by both passenger cars and trucks can be determined using the AASHTO relationship presented in Equation 1. The passing vehicle is assumed to accelerate at a constant rate (a) until the desired speed differential (m) with the passed vehicle is reached. Thus, t_1 can be calculated as m/a.
- The acceleration rate (a) and initial maneuver time (t_1) for passes by passenger cars as a function of design speed can be approximated by the AASHTO estimates in Table 1. Because of the lower performance capabilities of trucks, their acceleration rates during the initial maneuver are assumed to be half those used by passenger cars.
- The distance traveled in the left lane (d_2) can be estimated as

$$d_2 = V\left[\frac{2.93\,(V - m) + L_I + L_P - \dfrac{0.73m^2}{a}}{m}\right] \qquad (5)$$

This relationship is used in preference to the AASHTO expression for d_2 because it explicitly contains the lengths of the passing and passed vehicles (L_p and L_I) and the speed difference between the vehicles (m). It would be desirable to calibrate Equation 5 with field data.

- Equation 5 is based on the premise that the passing vehicle initially trails the passed vehicle by a 1-sec gap; it then returns to its normal lane leading the passed vehicle by a 1-sec gap. The passed vehicle is assumed to travel at constant speed and the passing vehicle is assumed to maintain an average speed differential equal to m during its occupancy of the left lane; the latter assumption is consistent with AASHTO policy, but more restrictive than the Glennon model (*12*), which assumes only that a speed differential equal to m is reached before the passing vehicle reaches the critical position.
- Passenger cars will accelerate when passing and maintain an average speed equal to the design speed of the highway, maintaining the same average speed differences used to derive Table 5. When passing, trucks are assumed to maintain only half of the speed difference of passenger cars, in keeping with the assumptions used to derive Table 6.
- The assumed lengths of passenger cars and trucks are 19 ft and 75 ft, respectively.

The sensitivity analysis results for the distance required to complete a pass are presented in Table 8 for the four passing scenarios considered previously—passenger car passing passenger car, passenger car passing truck, truck passing passenger car, and truck passing truck. The required passing distances for these four scenarios are illustrated in Figure 3. Except at very low speeds, all of the passing distances are much larger than the MUTCD minimum passing zone length of 400 ft.

Table 8 and Figure 3 show that in order to complete a passing maneuver at speeds of 60 mph or more under the stated assumptions, trucks require passing zones at least 2,000 ft long. There are relatively few such passing zones on two-lane highways, yet trucks regularly make passing maneuvers. The explanation of this apparent paradox is that, because there are very few locations where a truck can safely make a delayed pass, truck drivers seldom attempt them. Most passing maneuvers by trucks on two-lane highways are flying passes that require less passing sight distance and less passing zone length than delayed passes. Thus there may be no need to change current passing sight distance criteria to accommodate a truck passing a passenger car or a truck passing a truck as shown in Table 6. It makes little sense to provide enough passing sight distance for delayed passes by trucks when passing zones are not generally long enough to permit such maneuvers.

CONCLUSIONS

There is close agreement between the current MUTCD criteria for passing sight distance and the sight distance requirements for a passenger car passing another passenger car based on an analytical model recently developed by Glennon (*12*). Application of the Glennon model indicates that successively longer passing sight distances are required for a passenger car passing a truck, a truck passing a passenger car, and a truck passing a truck. There is no general agreement concerning which of these situations is the most reasonable basis for designing and operating two-lane highways. All of the passing sight distance criteria derived here are shorter than the

TABLE 8 PASSING ZONE LENGTH REQUIRED TO COMPLETE A PASS FOR VARIOUS PASSING SCENARIOS

Design or Pre-vailing Speed (mph)	Passing Vehicle Speed (V) (mph)	Speed Difference (m) Used by Passing Vehicle: Passenger Car	Speed Difference (m) Used by Passing Vehicle: Truck	Minimum Length of Passing Zone (ft): Passenger Car Passing Passenger Car	Passenger Car Passing Truck	Truck Passing Passenger Car	Truck Passing Truck
20	20	13	6.5	150	225	275	350
30	30	12	6	350	475	600	725
40	40	11	5.5	600	825	975	1,175
50	50	10	5	975	1,250	1,450	1,750
60	60	9	4.5	1,475	1,850	2,075	2,450
70	70	8	4	2,175	2,650	2,900	3,400

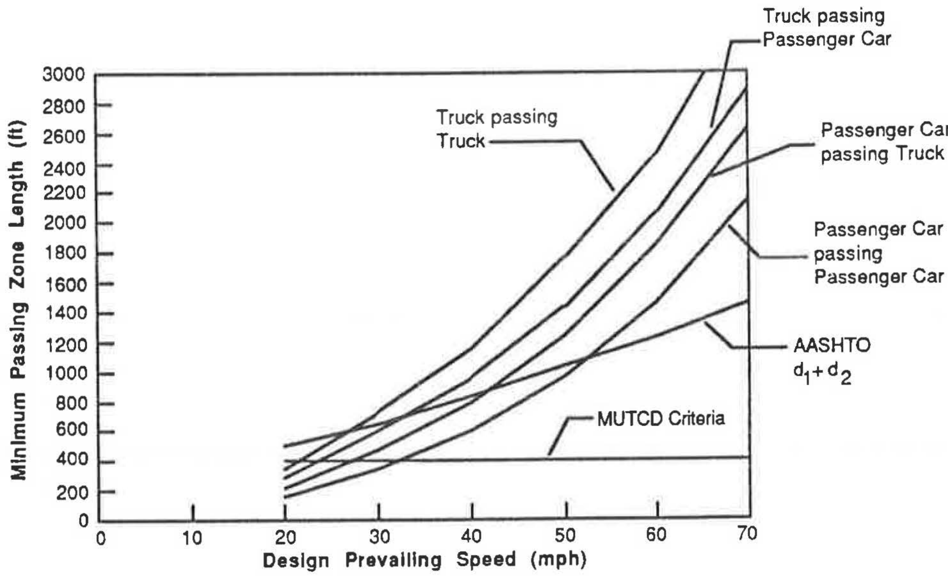

FIGURE 3 Minimum passing zone length to complete a pass at or near the highway design speed.

AASHTO design criteria, which are based on very conservative assumptions.

The analysis results indicate that, if a passenger car passing a passenger car is retained as the design situation, only minor modifications are needed to the MUTCD passing sight distance criteria. If a more critical design situation is selected (for example, a passenger car passing a truck), passing sight distances up to 250 ft longer than the current MUTCD criteria would be required. It is important to recognize that such a change in passing zone marking criteria would completely eliminate some existing passing zones and shorten others, even though passenger cars can safely pass other passenger cars in those zones. Clearly this would reduce the level of service on two-lane highways. This reduction in level of service would only be justified if there were demonstrated safety benefits. The current state-of-the-art of two-lane highway safety research has not addressed the question of whether there are such benefits. We do not know whether small increases in passing sight distance criteria will reduce accidents or whether passenger car drivers have more difficulty in judging the criticality of passing maneuvers when the passed vehicle is a truck rather than a passenger car. Research on these safety issues should be undertaken before any change is made in passing sight distance criteria to accommodate the sight distance requirements of passenger cars passing trucks.

The increased driver eye height of trucks partially, but not completely, offsets the increased passing sight distance requirements when the truck is the passing vehicle. Except at very sharp crests on high-speed highways, however, a truck can safely pass a passenger car on any crest where a passenger car can safely pass a truck. Thus the selection of the passenger car passing a truck as the design situation would, in most cases, also safely accommodate a truck passing a passenger car. There is great doubt about the wisdom of marking passing zones based on a truck as the passing vehicle, because it can be demonstrated that few passing zones on two-lane highways are long enough to accommodate delayed passes by trucks.

There are no current criteria for passing zone lengths, except for the default 400-ft guideline set by the MUTCD. For all design speeds above 30 mph, the distance required for one vehicle to pass another at or near that design speed is substantially longer than 400 ft, indicating a need for longer passing zones. Furthermore, there is research that indicates a higher rate of conflicts between passing and opposing vehicles in passing zones less than 800 ft in length. This research, together with the analyses in this paper, may justify an increase in minimum passing zone length to at least 800 ft for highways with a prevailing speed over 40 mph. The analyses in this paper also show that the required passing distances and passing zone lengths are increased substantially when the passing vehicle, the passed vehicle, or both, are trucks. Nevertheless, as in the case of passing sight distance criteria, there is no research that indicates whether there would be safety benefits from minimum passing zone lengths above 800 ft. Such research is needed because elimination of all passing zones shorter than the lengths shown in Table 8 could seriously degrade the level of service on two-lane highways.

This paper makes a strong case that Equations 3 and 4 provide a more reasonable representation of passing sight distance requirements on two-lane highways than either the current AASHTO or MUTCD criteria. Similarly, Equations 1 and 5 provide a realistic method for determining the distance required to make a delayed pass. These models follow more logically from the AASHTO assumptions concerning delayed passes than do either the AASHTO or MUTCD models. Furthermore, these models are sensitive to vehicle length in a way that the current AASHTO and MUTCD models are not. Given the explicit, quantitative estimates of passing sight distance and passing zone length requirements for different passing scenarios made in this paper, some readers may be disappointed that we have not made more specific recommendations for changes in current criteria. We lack sufficient data to make such recommendations. Neither our models nor the current AASHTO and MUTCD models have any direct, demonstrated relationship to the safety of passing maneuvers on two-lane highways. Such demonstrated safety relationships are needed before any change in passing and no-passing zone criteria can be reasonably contemplated.

ACKNOWLEDGMENT

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The findings and conclusions in this paper are those of the authors and do not necessarily represent the views of the Federal Highway Administration.

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Sight Distance Requirements for Trucks at Railroad-Highway Grade Crossings

KAY FITZPATRICK, JOHN M. MASON, JR., AND JOHN C. GLENNON

The sight distance requirements for large trucks at railroad-highway grade crossings are compared with current AASHTO policy. The key elements affecting sight distance requirements include driver characteristics such as perception-reaction time and vehicle characteristics such as vehicle speed, length, acceleration, and braking distances. The results from sensitivity analyses are compared with current policy and are summarized for each sight distance consideration. The findings imply that current criteria for sight distance along the highway and along the tracks for a moving highway vehicle may not be adequate for large trucks. In contrast, the current AASHTO values for sight distance along the tracks for a stopped highway vehicle adequately reflect current truck performance capabilities.

The 1986 FHWA *Railroad-Highway Grade Crossing Handbook* (*1*) states that railroad-highway grade crossings are unique in that they are the intersection of two transportation modes. These modes differ both in the physical characteristics of their traveled ways and in their vehicle operations. A railroad-highway grade crossing may be viewed as a special type of highway intersection, with the three basic elements of highways present: the driver, the vehicle, and the physical intersection. As with a highway intersection, drivers must appropriately yield the right-of-way to intersecting traffic; unlike highway intersections, the intersecting traffic—trains—does not yield the right-of-way. Drivers of motor vehicles have the flexibility to change their path of travel and can change their speed within a relatively short distance. Locomotive engineers are restricted to moving their trains down a fixed path and require relatively long distances and times to change speed. Because of this, drivers need adequate clear sight triangles to avoid collisions with trains.

This paper includes both a critical review of the current procedures and a sensitivity analysis to determine the sight distance requirements for the kinds of trucks permitted by the 1982 Surface Transportation Assistance Act (STAA), which are not currently included in the AASHTO Green Book. No changes in the general design procedure are recommended on the basis of this analysis. It does, however, provide specific information on the effects of current physical and performance characteristics of trucks on sight distance requirements.

K. Fitzpatrick and J. M. Mason, Jr., Pennsylvania Transportation Institute, Pennsylvania State University, University Park, Pa. 16802. J. C. Glennon, John C. Glennon, Chartered, 8340 Mission Rd., Suite B-12, Prairie Village, Kans. 66206.

CURRENT RAILROAD-HIGHWAY GRADE CROSSING SIGHT DISTANCE POLICY

Both the FHWA Handbook (*1*) and the AASHTO Green Book (*2*) use the same principles for determining safe sight triangles at railroad-highway grade crossings. They both consider sight distance requirements for a moving highway vehicle and for a highway vehicle accelerating from a stop at the crossing, as shown in Figure 1. For the moving-vehicle situation, the sight distance (d_H) along the highway must, as a minimum, be the safe stopping sight distance for the given

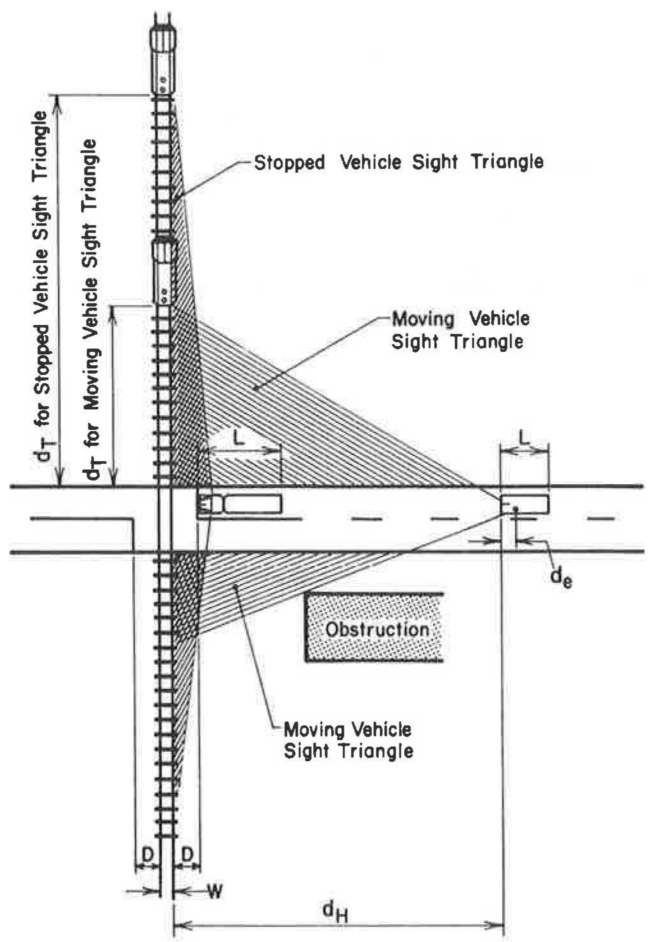

FIGURE 1 Dimensions considered in railroad-highway grade crossing sight distance (*1*).

approach speed. The sight distances along the track for this situation are the distances traveled by the train during the time the highway vehicle traverses both the highway distance (d_H) and the distance to clear the crossing. For the stopped-vehicle situation, the highway vehicle starts from a minimum safe distance from the crossing. The distances along the track for this situation are those traveled by the train at various speeds while the highway vehicle accelerates and just clears the crossing.

Sight Distance Along the Highway for a Moving Vehicle

The minimum sight distance measured along the highway (d_H) is from the nearest rail to the driver of a vehicle. It is the sum of the minimum stopping sight distance and the minimum clearance distance between the tracks and the driver after the vehicle stops. This distance allows an approaching vehicle to avoid collision by stopping without encroaching on the crossing area. The minimum sight distance formula used in the FHWA Handbook (*1*) and the AASHTO Green Book (*2*) is

$$d_H = 1.47V_v t_{pr} + \frac{V_v^2}{30f} + D + d_e \tag{1}$$

where

d_H = sight distance along the highway (ft),
V_v = velocity of vehicle (mph),
t_{pr} = perception/reaction time of driver (sec) (assumed: $t_{pr} = 2.5$ sec),
f = coefficient of friction used in braking (see Table 1 for assumed values),
D = clearance distance from front of vehicle to the nearest rail (ft) (assumed: $D = 15$ ft), and
d_e = distance from driver to the front of vehicle (ft) (assumed: $d_e = 10$ ft).

The coefficient of friction values f are from the AASHTO Green Book criteria for stopping sight distance. These values, the result of several studies cited in the Green Book, represent the marginal deceleration rates for a passenger car in locked-wheel braking on a wet pavement.

Sight Distance to and Along Tracks for a Moving Vehicle

The legs of the clear "approach sight triangle" are formed by d_H, the distance of the vehicle from the track, and d_T, the distance of the train from the crossing. The equation for d_H is discussed above. The minimum distance along the track (d_T) is from the nearest edge of the highway travel lane being considered to the front of the train. It is the product of the train speed and the time required by the highway vehicle to both traverse the highway leg (d_H) and clear the crossing. The distance formula used in both the FHWA Handbook and the AASHTO Green Book is

$$d_T = \frac{V_t}{V_v}\left(1.47\ V_v t_{pr} + \frac{V_v^2}{30f} + 2D + L + W\right) \tag{2}$$

TABLE 1 COEFFICIENTS OF FRICTION, f (*2*)

Speed (mph)	f
10	0.40
20	0.40
30	0.35
40	0.32
50	0.30
60	0.29
70	0.28

where

d_T = sight distance along the railroad tracks for a moving vehicle (ft),
V_t = velocity of train (mph),
V_v = velocity of vehicle (mph),
t_{pr} = perception-reaction time of vehicle (sec) (assumed: $t = 2.5$ sec),
f = coefficient of friction used in braking (see Table 1 for assumed values),
D = clearance distance from the vehicle to the nearest rail (ft) (assumed: $D = 15$ ft),
L = length of vehicle (ft) (assumed: $L = 65$ ft), and
W = distance between outer rails (ft) (assumed for a single track: $W = 5$ ft).

The FHWA Handbook and AASHTO Green Book assume a 65-ft truck crossing a single track at 90 degrees on a flat terrain. The coefficient of friction values assumed are those in Table 1. Cautions are offered that adjustments should be made for unusual vehicle lengths and acceleration capabilities, as well as for multiple tracks, skewed crossings, and grades.

Sight Distance Along Tracks for a Stopped Vehicle

The third sight distance consideration is the sight triangle needed to allow a stopped vehicle to accelerate and cross the tracks before the train reaches the crossing. It includes the perception-reaction time of the driver and vehicle characteristics such as maximum speed of vehicle in starting gear, acceleration capability of vehicle, and length of vehicle. The required distance (d_T) along the tracks is determined in the FHWA Handbook and AASHTO Green Book as

$$d_T = 1.47V_t\left(\frac{V_g}{a_1} + \frac{L + 2D + W - d_a}{V_g} + J\right) \tag{3}$$

where

d_T = sight distance along the railroad tracks for a stopped vehicle (ft),
V_t = velocity of train (mph),
V_g = maximum speed of vehicle in first gear (fps) (assumed: $V_g = 8.8$ fps),
a_1 = acceleration of vehicle in first gear (fpsps) (assumed: $a_1 = 1.47$ fpsps),
L = length of vehicle (ft) (assumed: $L = 65$ ft),
W = distance between outer rails (ft) (assumed for a single track: $W = 5$ ft),

D = clearance distance from front of vehicle to the nearest rail (ft) (assumed: D = 15 ft),
J = sum of perception-reaction time of driver and time required to activate the clutch or an automatic shift (sec) (assumed: J = 2.0 sec), and
d_a = distance vehicle travels while accelerating to maximum speed in first gear (ft) = $V_g^2/2a_1$.

The FHWA Handbook and the AASHTO Green Book also assume a 65-ft truck crossing a single track at 90 degrees on a flat terrain for this procedure. Adjustments should be made for longer vehicle lengths, slower acceleration capabilities, multiple tracks, skewed crossings, and other than flat highway grades.

CRITIQUE OF POLICY

A review of driver characteristics by McGee et al. (3) addressed changes in sight distance requirements that accompany changes in driver characteristics. The driver characteristic reviewed for railroad-highway grade crossing sight requirements (as presented in the first edition of the FHWA Handbook) was perception-reaction time. Their findings indicate that the sight requirements are relatively insensitive to a change in the perception-reaction time.

A review of the cases in the AASHTO Green Book by McGee et al. (4) found the formulation for calculating the minimum corner sight triangle for a moving vehicle to be correct and reasonable. They also found that the concept for determining the minimum sight distance along a track for a vehicle at a stopped position was correct. The concept adequately considers both the driver and vehicle requirements.

Wilde et al. (5) reported that the lack of uniformity in driver behavior indicates a high level of uncertainty concerning the correct response to grade crossings, and that this may be a major cause of crossing accidents. Vehicle speed variations were higher as the distance to the crossing decreased. Specific speed variations for trucks were not reported.

Schoppert and Hoyt (6) identified factors influencing safety at railroad-highway grade crossings in their NCHRP report. They reviewed a sample of 3,627 accidents: one-third involved trains, one-third occurred when the train was present but not involved, and one-third occurred when the train was not even present. They found the following:

- The distribution of vehicle speeds at the crossing differs from that along the highway prior to the influence of the crossing. These conditions were believed to contribute significantly to multiple-vehicle accidents at crossings.
- Trucks were involved in accidents with trains relatively more frequently than other vehicles. This statistic makes a strong argument for using truck design values for sight distance calculations.
- High truck involvement in accidents may be attributable to the truck's greater length, which causes it to occupy the crossing longer.

AASHTO stopping sight distance criteria (2) use coefficients of friction that are intended to represent the deceleration rates used by a passenger car in locked-wheel braking on a wet pavement. Trucks cannot safely make a locked-wheel stop without the risk of losing control of the vehicle. A discussion of braking distances by Harwood et al. (in this Record) shows that the deceleration rates used by trucks to make controlled stops are generally lower than the deceleration rates used by passenger cars making locked-wheel stops.

The FHWA Handbook does not cite the studies used as the basis for the following assumptions:

- The speed of the vehicle in selected starting gear is 8.8 fps, and
- The acceleration of the vehicle in starting gear is 1.47 fpsps.

Nevertheless, the use of these assumptions for computing the sight distance along the tracks for a stopped vehicle appears to be reasonable.

SENSITIVITY ANALYSIS

The current sight distance policies directly or indirectly use different vehicle types as the design vehicle. By using deceleration rates for a passenger car in locked-wheel braking on a wet pavement, sight distance along the highway for a moving vehicle is derived with a passenger car as the design vehicle. The derivation for sight distances along the tracks for a moving vehicle mixes design vehicle characteristics by using passenger car deceleration rates but a 65-ft vehicle length (typical for a WB-60 truck). The design vehicle for sight distance along tracks for a stopped vehicle is a 65-ft truck, with reasonable assumptions for both acceleration and the maximum speed in first gear.

Following is a sensitivity analysis to determine the railroad-highway sight distance requirements for the types of trucks permitted since the 1982 STAA, which are not currently included in the AASHTO Green Book. This sensitivity analysis is a simple extension of the existing sight distance considerations to reflect current truck characteristics and performance. Table 2 presents the equations derived and the parameters currently used in the three sight distance considerations. They include a driver-related characteristic (perception-reaction time) and vehicle-related characteristics (stopping sight distance, vehicle length, and maximum speed and acceleration of vehicle in first gear). Table 3 contains the values of the vehicle-related parameters (including vehicle length, stopping sight distance, and vehicle acceleration) that have been varied in the sensitivity analysis.

Truck lengths of 70 and 75 ft were used in the analyses. An overall length of 70 ft represents a STAA tractor-semitrailer truck with a 53-ft trailer unit. The overall length of 75 ft represents a STAA "double bottom" truck with a conventional cab-behind-engine tractor and two 28-ft trailers.

The stopping sight distances used are those derived by Harwood et al. (in this Record), based on estimates of truck braking distances developed by Fancher (7). These distances represent controlled braking by an empty truck on a poor, wet road with relatively good radial tires (at least 12/32 in of tread depth). The truck braking performance of drivers varies widely as a result of driver expertise. This variation exists because many truck drivers lack experience in emergency

TABLE 3 SUMMARY OF PARAMETERS VARIED IN SENSITIVITY ANALYSIS FOR RAILROAD-HIGHWAY GRADE CROSSING SIGHT DISTANCE

Consideration	Vehicle Length (ft)	Stopping Sight Distance (SSD)			Additional Assumptions
Sight distance along a highway	NA	Truck Driver Performance			NA
		Speed	Worst	Best	
$d_H = SSD + D + d_e$		20 mph	150'	125'	
		30 mph	300'	250'	
$d_H = SSD + 10 + 15$		40 mph	500'	375'	
		50 mph	725'	525'	
$d_H = SSD + 25$		60 mph	975'	700'	
		70 mph	1275'	900'	
Sight distance to and along tracks for a moving vehicle	70' tractor semi-trailer truck	Truck Driver Perforance			NA
	75' tractor semi-trailer-full trailer truck (double bottom)	Speed	Worst	Best	
		20 mph	150'	125'	
		30 mph	300'	250'	
		40 mph	500'	375'	
$d_T = \frac{V_t}{V_v}(SSD + 2D + L + W)$		50 mph	725'	525'	
$d_T = \frac{V_t}{V_v}(SSD + 2 * 15 + L + 5)$		60 mph	975'	700'	
$d_T = \frac{V_t}{V_v}(SSD + 35 + L)$		70 mph	1275'	900'	
Sight distance along tracks for a stopped vehicle	70' tractor semi-trailer truck	NA			t_c = time to clear hazard zone (from Gillespie's equation (9))
	75' tractor semi-trailer-full trailer truck (double bottom)				
$d_T = 1.47V_t(t_c + J)$					
$d_T = 1.47V_t\,[0.682 * (2 * D + W + L)/Vmg + 3.0 + 2.0]$					
$d_T = 1.47V_t\,[0.682 * (2 * 15 + 5 + L)/8 + 3.0 + 2.0]$					
$d_T = V_t\,[0.125L + 11.73]$					

TABLE 2 SUMMARY OF PARAMETERS FOR RAILROAD-HIGHWAY GRADE CROSSING SIGHT DISTANCE

Equations	Perception-Reaction Time (sec)	Stopping Sight Distance (SSD)		d_e (ft)	D (ft)	W (ft)	Length of Vehicle (ft)	V_g (fps)	a_1 (fpsps)
		SSD Values(2)							
Sight distance		Speed	SSD						
along a highway	$t_{pr} = 2.5$	20 mph	125'	10	15	NA	NA	NA	NA
		30 mph	200'						
$d_H = SSD + D + d_e$		40 mph	325'						
		50 mph	475'						
$d_H = SSD + 10 + 15$		60 mph	650'						
		70 mph	850'						
$d_H = SSD + 25$									
		SSD Values(2)							
Sight distance to and		Speed	SSD						
along tracks for a	$t_{pr} = 2.5$	20 mph	125'	NA	15	5		NA	NA
moving vehicle		30 mph	200'						
		40 mph	325'						
		50 mph	475'						
$d_T = \frac{V_t}{V_v}[SSD + 2D + L + W]$		60 mph	650'				65 WB-60		
		70 mph	850'						
$d_T = \frac{V_t}{V_v}[SSD + 2 * 15 + 65 + 5]$									
$d_T = \frac{V_t}{V_v}[SSD + 100]$									
Sight distance along tracks for a stopped vehicle	$J = 2.0$	NA		NA	15	5		8.8	1.47
$d_T = 1.47V_t\left[\frac{V_G}{a_1} + \frac{L + 2D + W - d_a}{V_g} + J\right]$ $d_a = \frac{V_g^2}{2a_1}$							65 WB-60		
$d_T = 1.47V_t\left[\frac{8.8}{1.47} + \frac{65 + 2 * 15 + 5 - (8.8^2/(2 * 1.47))}{8.8} + 2.0\right]$									
$d_T = 16.9V_t$									

braking, and because different drivers accept varying amounts of "risk" in what is potentially a hazardous operation that could lead to truck jackknifing. Fancher (*7*) found that the worst-performing driver has a braking efficiency of approximately 62 percent of the vehicle capability, while the best-performing drivers can achieve nearly 100 percent of the vehicle capability. A range of stopping sight distances appropriate for both the worst and best drivers (62 to 100 percent driver control efficiency) is considered in this paper.

Since truck size, weight, and performance characteristics have been changing, more recent truck acceleration information is needed. In 1986, Gillespie (*8*) reported clearance times for trucks crossing an intersection. The time (t_c) required for a truck to clear a hazard zone starting from a full stop and remaining in initial gear during the maneuver was estimated by the following equation:

$$t_c = 0.682 \frac{L_{HZ} + L}{V_{mg}} + 3.0 \text{ sec} \quad (4)$$

where

L_{HZ} = length of the hazard zone (ft) = $2D + W$,
D = clearance distance from front of vehicle to the nearest rail (ft) (assumed: D = 15 ft),
W = distance between outer rails (ft) (assumed for a single track: W = 5 ft),
L = length of the truck (ft), and
V_{mg} = maximum speed in a selected gear (mph) (determined by Gillespie as 8 mph for a level surface).

Equation 4 assumes that the gear design, engine speed, and the tire size are such that the truck's maximum speed is 60 mph on a level surface. It also assumes that a truck will remain in the initial gear without shifting while negotiating the hazard zone. Gillespie also developed a maximum speed in initial gear versus grade curve for determination of clearance time for trucks accelerating on a grade.

Sight Distance Along Highway for a Moving Vehicle

The sight distance along the highway to the crossing (d_H) increases significantly in comparison with the current FHWA criteria when the increased stopping sight distances of trucks are considered. Table 4 presents the required sight distances for current criteria in comparison with trucks with the worst-performing and best-performing drivers. (The stopping sight distances for these drivers are shown in Table 3.) The results shown in Table 4 are shown depicted in Figure 2. Although the effect appears minimal for a truck with the best-performing driver (between 7 to 22 percent increase in sight distance), significant increases in sight distances are required for a truck with the worst-performing driver (between 30 and 54 percent increase).

Sight Distance Along Tracks for a Moving Vehicle

A sensitivity analysis of the sight distance requirements along the track from the crossing (d_T) found similar results (see Table 5 and Figure 3 for a 70-ft truck length). A 70-ft truck requires a 23 percent increase in sight distance at 20 mph and up to a 47 percent increase at 70 mph for a worst-performing driver. A best-performing driver of a 70-ft truck requires a maximum of a 20 percent increase in sight distance. A 75-ft

TABLE 4 SENSITIVITY ANALYSIS FOR SIGHT DISTANCE ALONG A HIGHWAY (d_H)

	VEHICLE SPEED, V_V (mph)					
	20	30	40	50	60	70
	d_H (ft)	d_H (ft)	d_H (ft)	d_H (ft)	d_H (ft)	d_H (ft)
Current values	135	225	340	490	660	865
Sight Distances for a truck with worst-performance driver	175	325	525	750	1000	1300
Sight Distances for a truck with best-performance driver	150	275	400	550	725	925

Note: FHWA rounded all calculated distances up to the next higher 5-foot increment.

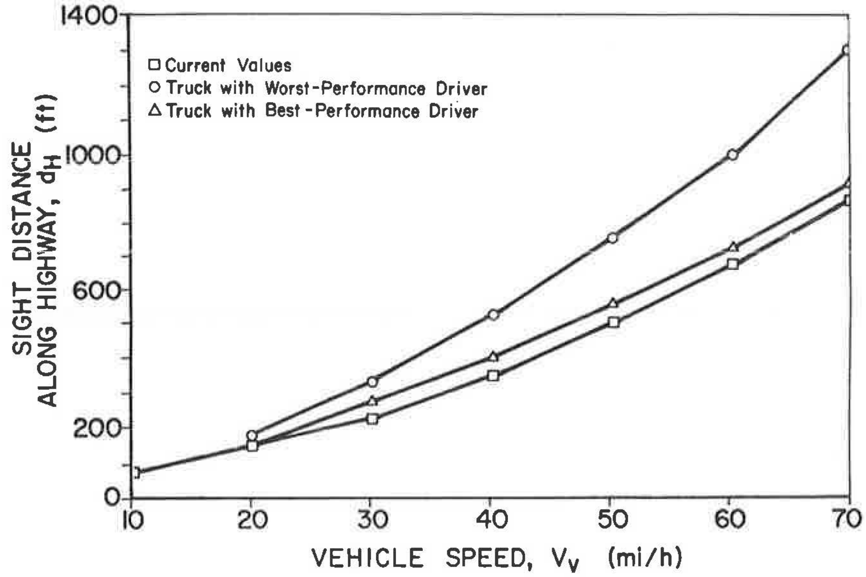

FIGURE 2 Sensitivity analysis for sight distance along highway (d_H).

truck requires similar increases in sight distance (a maximum 22 percent increase for a best-performing driver and up to a 49 percent increase for a worst-performing driver). Not only did the greater truck length increase the required sight distance, but the braking distance for the worst driver for both truck lengths also significantly increased the required sight distance.

Sight Distance Along Tracks for a Stopped Vehicle

The sight distance requirement along the tracks for a stopped vehicle is not very sensitive to vehicle length. Table 6 and Figure 4 present the results of increasing the current design vehicle length of 65 ft to 70 and 75 ft and using the equation developed by Gillespie (*8*) for the time to clear a hazard zone (Equation 4 in this paper). The sight distance values calculated using AASHTO assumptions—of a 65-ft truck, 8.8 fps for maximum speed of vehicle in first gear, and 1.47 fpsps for acceleration of vehicle in first gear—are longer than those calculated using a 70- or 75-ft truck length and the Gillespie model for clearance times (t_c). This is the result of the Gillespie model providing lower values of clearance times (t_c) than the current AASHTO criteria.

SUMMARY OF FINDINGS

The sensitivity analyses demonstrate that trucks moving ahead to the railroad-highway grade crossings require increased sight distance along the highway (d_H) primarily because of their longer braking distances. The added sight distance requirements are substantial for trucks with the worst-performing driver, but are minimal for trucks with the best-performing driver.

Similar conclusions were reached for sight distance needed along the tracks from the crossing (d_T) for a moving vehicle. Substantially longer sight distances are required for a truck with the worst-performing driver (up to 49 percent increase in sight distance).

In contrast, the current requirements for sight distance required along the tracks for a stopped vehicle, based on a 65-ft truck, were found to be adequate for the 70- and 75-ft trucks when the Gillespie (*8*) model for clearance time is used.

CONCLUSIONS

The sight triangles required for highway vehicles approaching a railroad-highway grade crossing are considerably larger than those required by the FHWA Handbook and the AASHTO Green Book if the needs of truck drivers are fully considered. Currently, with a minimum FHWA or AASHTO sight triangle for assumed speeds, a truck driver can easily face a dilemma. If a train appears at the track apex of this sight triangle as the truck reaches the highway apex, the truck driver must decide to proceed at a constant or increased speed rather than either slowing or stopping. If he decides to stop, he will collide with the train before coming to a full stop. If he begins to stop and then decides to proceed, he will collide with the train before clearing the crossing.

To provide an adequate margin of safety for truck drivers at railroad-highway grade crossings, current FHWA (*1*) and AASHTO (*2*) sight triangle values for moving vehicles should be increased to allow for longer trucks and for some measure of the greater stopping distances of trucks compared with passenger vehicles. To arrive at representative values, a decision regarding the level of truck driver performance is required. The range of values calculated earlier should provide some guidance for this task.

Considering that bigger sight triangles may be necessary to accommodate large trucks at railroad-highway grade cross-

TABLE 5 SENSITIVITY ANALYSIS FOR SIGHT DISTANCE TO AND ALONG TRACKS (d_T)

Train Speed V_t (mph)	Vehicle Speed, V_v (mph)					
	20	30	40	50	60	70
	d_T (ft)	d_T (ft)	d_T (ft)	d_T (ft)	d_T (ft)	d_T (ft)
CURRENT AASHTO PROCEDURES USING A 65-FOOT TRUCK						
10	105	100	105	115	125	135
20	210	200	210	225	245	270
30	310	300	310	340	370	405
40	415	395	415	450	490	540
50	520	495	520	565	615	675
60	620	595	620	675	735	810
70	725	690	725	790	860	940
80	830	790	830	900	980	1075
90	930	930	930	1010	1105	1210
SIGHT DISTANCES FOR A 70-FOOT TRUCK WITH WORST-PERFORMANCE DRIVER						
10	128	135	151	166	180	197
20	255	270	303	332	360	394
30	383	405	454	498	540	591
40	510	540	605	664	720	789
50	638	675	756	830	900	986
60	765	810	908	996	1080	1183
70	893	945	1059	1162	1260	1380
80	1020	1080	1210	1328	1440	1577
90	1148	1215	1361	1494	1620	1774
SIGHT DISTANCES FOR A 70-FOOT TRUCK WITH BEST-PERFORMANCE DRIVER						
10	115	118	120	126	134	144
20	230	237	240	252	268	287
30	345	355	360	378	403	431
40	460	473	480	504	537	574
50	575	592	600	630	671	718
60	690	710	720	756	805	861
70	805	828	840	882	939	1005
80	920	947	960	1008	1073	1149
90	1035	1065	1080	1134	1208	1292

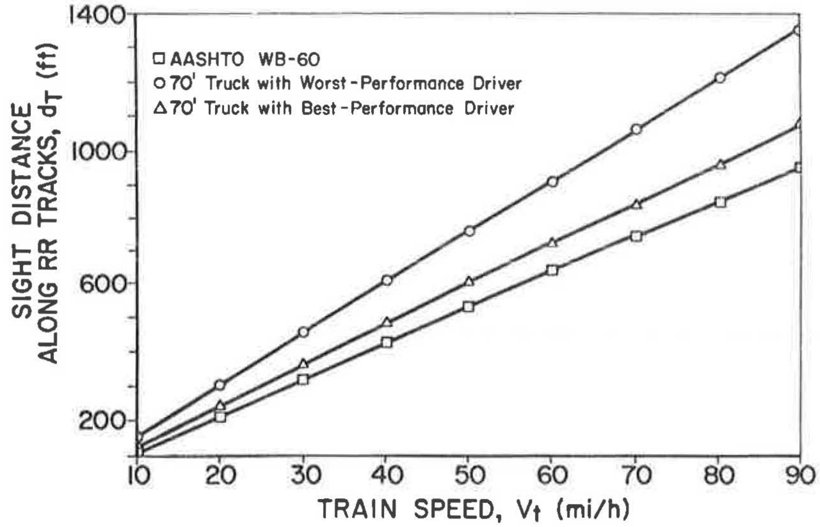

FIGURE 3 Sensitivity analysis for sight distance along tracks (d_T) for a moving vehicle at 40 mi/h.

TABLE 6 SENSITIVITY ANALYSIS FOR SIGHT DISTANCE ALONG TRACK FOR A STOPPED VEHICLE

TRAIN SPEED V_t (mph)	AASHTO PROCEDURE AASHTO WB-60 TRUCK d_T (ft)	70' TRACTOR-SEMI TRAILER TRUCK d_T (ft)	75' TRACTOR-SEMI TRAILER-FULL TRAILER TRUCK d_T (ft)
10	240	206	212
20	481	412	423
30	721	617	635
40	962	823	847
50	1202	1029	1058
60	1443	1235	1270
70	1683	1441	1482
80	1924	1646	1693
90	2164	1852	1905

ASSUMED:

t_c determined from Gillespie's equation

t_c = 12.0 seconds for 70' truck

t_c = 12.4 seconds for 75' truck

L_{hz} = 2D + W = 2*15' + 5' = 35'

V_{mg} = 8.0 mph

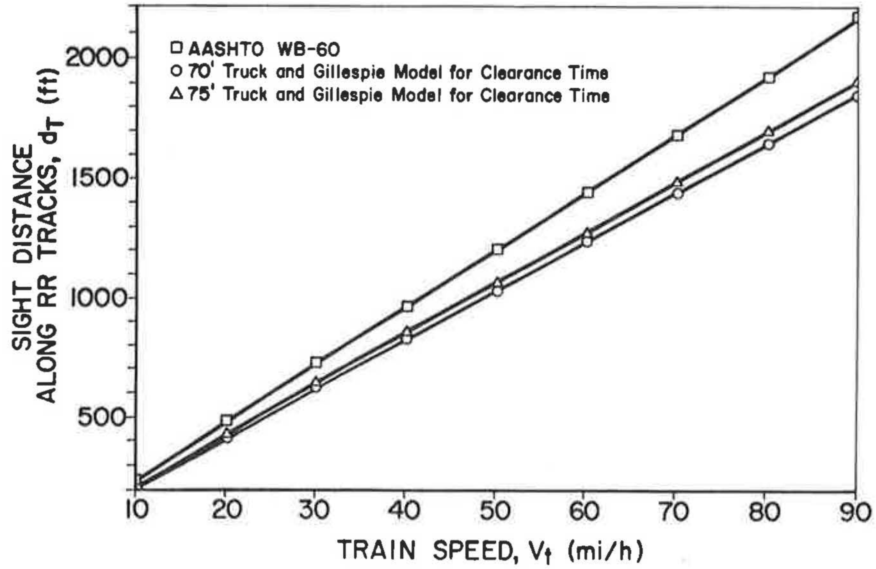

FIGURE 4 Sensitivity analysis for sight distance along tracks for a stopped vehicle (d_T).

ings, many more crossings than previously thought may have physical constraints that make the available sight triangle unacceptable. Therefore, if the needs of truck drivers are truly considered, the need for positive and active traffic controls at grade crossings may be greater than previously thought.

In contrast to the moving-vehicle analysis described above, the current FHWA and AASHTO sight distance criteria for stopped vehicles appear to adequately accommodate the needs of truck drivers.

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Horizontal Sight Distance Considerations in Freeway and Interchange Reconstruction

JOEL P. LEISCH

With improvements being made to freeways and expressways, the problem of inadequate stopping sight distance on curves accompanying installation of a concrete barrier may arise. This could also occur when lanes are added in the median of a freeway or expressway and a narrow median results. It also may become a problem on curved ramps on structures where the parapet may not be offset sufficiently from the traveled-way. One solution to the problem for freeway medians is to provide a wider shoulder (greater offset to barrier) for the traveled-way turning to the left. Where a constant median width exists, the left shoulder for the opposing direction is, of course, narrowed. When a ramp is on a structure (bridge or retaining wall) and there is a curve to the left, the traveled-way can be shifted to the right, providing a wider left shoulder for sight distance and breakdown. The right shoulder would then be narrowed, preferably to not less than 4 ft. It must be kept in mind that stopping sight distance is only one of many design and operational considerations in planning an improvement. There certainly can be trade-offs with other features and the potential influence on accident experience. Further study is needed to ascertain the optimum dimensions of all cross-sectional elements to best satisfy safety, operational, and design requirements.

As improvements are made to freeways and expressways, the problem of inadequate stopping sight distance on curves accompanying the installation of a concrete barrier may arise. This could also occur when lanes are added in the median of a freeway or expressway and a narrow median results. It also may become a problem on curved ramps on structures where the parapet may not be offset sufficiently from the traveled-way.

Many designers are not aware of these possible situations (combination of curvature and shoulder width), which may not only occur where shoulder width is less than 10 ft but in some cases can become a problem when a full shoulder is provided. Figure 1 [taken from Figure III-25A of AASHTO's 1984 Green Book (*1*)] clearly shows that when the curvature exceeds approximately 50–70 percent of the maximum curve for a specific design speed and there is a 10-ft shoulder, stopping sight distance for that design speed is not provided. This also holds true for a curve to the right where a barrier or parapet is placed adjacent to the right shoulder.

How can a designer deal with these situations or with the preparation of a design with constrained conditions? Presented here are examples of the situations mentioned above

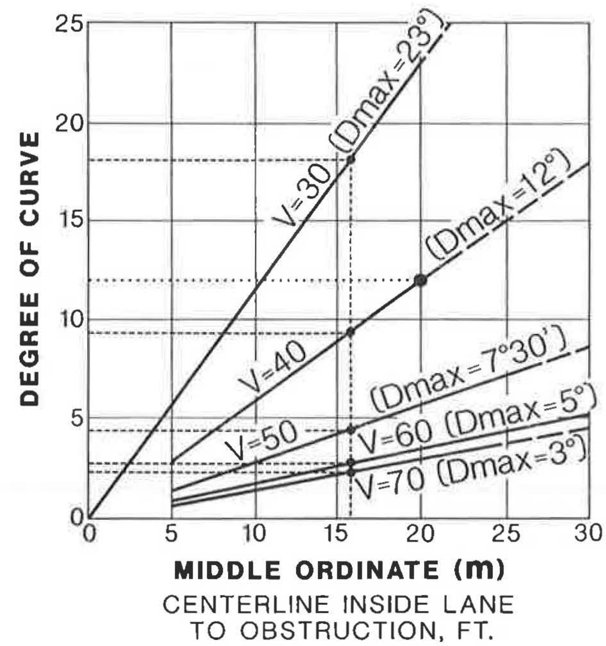


FIGURE 1 Curvature and middle ordinate relationships for lower values of stopping sight distance.

and sample solutions. In addition, two examples from the field are shown and the applied solutions described.

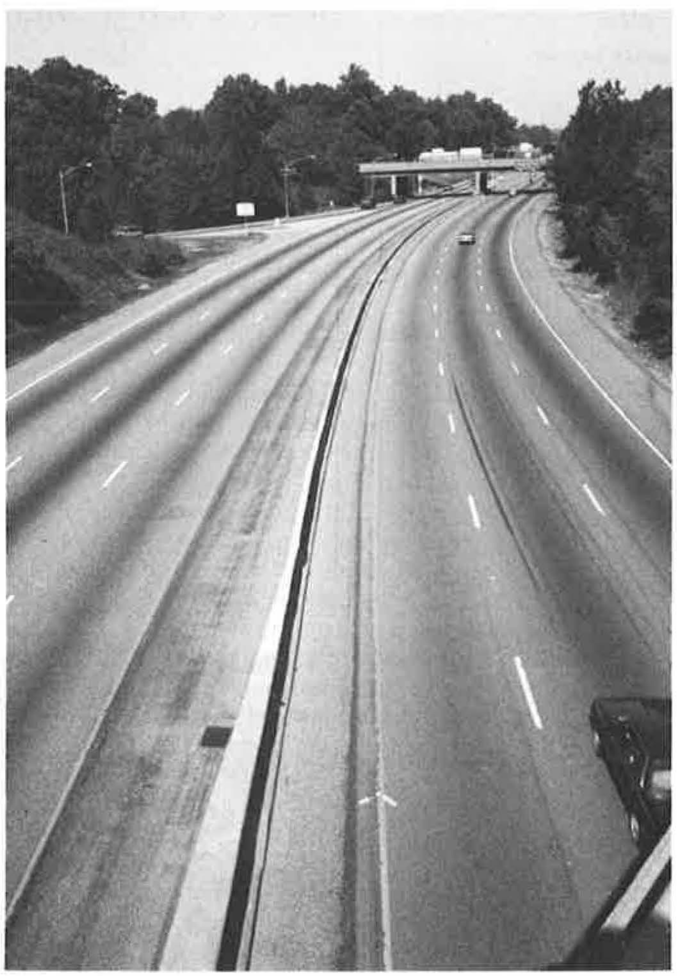
FREEWAY MEDIAN

The median barrier often restricts sight distance, as previously mentioned, even when a 10-ft shoulder is provided where the curve exceeds 2 degrees for a 70 mph design and 2.5 degrees for a 60 mph design. The maximum curvatures are approximately 3 degrees and 5 degrees, respectively, for 70 mph and 60 mph design speeds using a .08 ft/ft maximum superelevation. Consequently, numerous situations may already exist in urban areas where there is inadequate stopping sight distance. Where medians are narrower than 22–24 ft, the problem may occur more frequently.

One solution to this potential problem is to provide a wider shoulder (greater offset to barrier) for the traveled-way turning to the left. Where a constant median width exists, the left shoulder for the opposing direction is, of course, narrowed. It would not, however, be advisable to reduce this shoulder to less than 6 ft. Figure 2 shows this solution.

Jack E. Leisch & Associates, 1603 Orrington Ave., Suite 1200, Evanston, Ill. 60201.

SHIFTED MEDIAN BARRIER—FROM DRIVER'S PERSPECTIVE



SHIFTED MEDIAN BARRIER AT THE END OF THE CURVE FROM ABOVE

FIGURE 2 Shifted median barrier—two views.

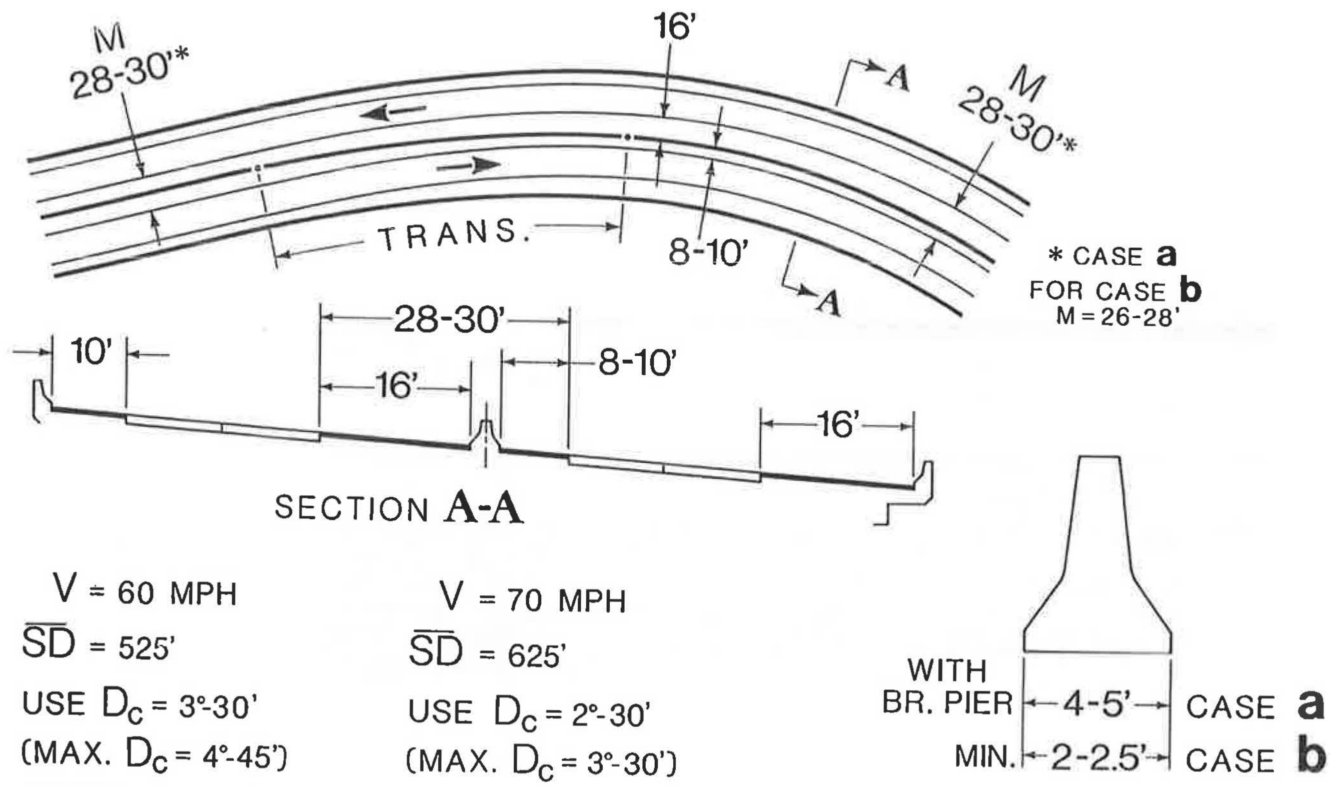


FIGURE 3 An example of freeway sight distance with a median barrier.

This design resulted from the addition of a median-side lane in each direction of travel. The median became 20 ft in width as a result of the widening. Through this curve (shown in Figure 2) the shoulder is 11 ft for the traveled-way curving to the left, and a 7-ft shoulder is provided for the opposing roadway. This solution provides adequate stopping sight distance for the critical direction of travel. The 7-ft shoulder for the opposing direction, while not a full shoulder, would provide a partial refuge for a stalled vehicle with minimal influence on vehicles operating in the left lane.

Utilizing the concept as implemented in the field and described above, two examples are presented below. These examples fully comply with the guidelines for sight distance and shoulder width presented in the AASHTO Policy for new design or major reconstruction projects (refer to Figure 3).

Both examples are for median widths of 26 to 30 ft. One is for a design speed of 60 mph and a curvature of 3.5 degrees and the other is for a design speed of 70 mph and a curvature of 2.5 degrees. The required offset to the face of the barrier from the edge of the traveled-way is 16 ft in both cases. Note that in the figure the barrier has been transitioned upon entry into the curve from its center position in the median to the necessary 16-ft offset through the curve. The resultant shoulder for the opposing traveled-way is 8 to 10 ft, depending on median and barrier width. It is also possible that stopping sight distance restrictions may occur on the right side of the freeway traveled-way on curves to the right when a barrier, parapet, or retaining wall is adjacent to the shoulder. This is also shown in Figure 3. As can be seen in the cross-section and plan view, the critical shoulder has been widened to 16 ft through the curve to provide for safe stopping sight distance.

RAMPS ON A STRUCTURE OR RETAINING WALL

Ramps can also have sight distance problems on curves when the ramp is on a bridge or adjacent to a retaining wall. As with the mainline traveled-way, this can occur on curves both to the left and to the right.

Referring to Figure 1, it can be seen that for a ramp with a 40 mph design speed and a maximum curvature of 12 degrees, a midordinate offset to the barrier is 20 ft from the center of the lane. Using a 10-ft ramp traveled-way, the offset from the edge of traveled-way to obstruction, barrier, or retaining wall is 12 ft. Most agencies presently use a 10-ft right shoulder on the right—this is inadequate for the conditions described above for a curve to the right. The 10-ft shoulder would be adequate for an 11 degree curve and a 40 mph design speed.

With a curve to the left (40 mph design speed), the typical 2-ft, 4-ft, or 6-ft left shoulder on a ramp would permit curves of approximately 6, 7, and 8 degrees, respectively, to provide for stopping sight distance. The left curve and the narrower left shoulder would apparently become critical in more cases than the right-curve, right-shoulder condition. With existing ramps or ramps to be constructed where this sight distance restriction already exists or might occur, a relatively simple solution may be effective. Shifting the ramp traveled-way to the left or right within the total pavement width could produce

FIGURE 4 Curve to the left and ramp traveled-way shifted right.

the desired offset to the obstruction, parapet, or retaining wall. An example of this is shown in Figure 4. In this actual situation, the 28-ft parapet-to-parapet width has been used to achieve safe stopping sight for this left-turning ramp onto a bridge. As can be seen, the left shoulder is 10 ft and the right shoulder is 2 ft. This was accomplished by restriping the pavement edges through the curve to produce the adequate offset to the parapet. An important feature of the design solution is that a shoulder or refuge area is still provided. This solution can be easily accomplished on existing ramps with adequate total width and can be considered on new interchange ramps as well.

Figures 5 and 6 show possible solutions for curves to the left and right. The first of these (curve to the left) is a ramp with a 40 mph design and a total width of 30 ft. The left curve is 9.5 degrees, resulting in an 8-ft offset to the parapet. As can be seen, the 16-ft traveled-way is transitioned 4 ft to the right to provide this offset.

Figure 6 illustrates a curve to the right where the ramp, with a 40 mph design speed and a 12 degree curve, requires a 12-ft shoulder or offset to the parapet or barrier. In this case, the traveled-way is transitioned 2 ft left upon entering the curve to produce the desired offset.

The above solutions, as in the actual cases illustrated, are simple to accomplish. Other types of solutions are also possible, which may include widening the total pavement and bridge width if full shoulders are to be maintained both left and right. The conditions, however, should be analyzed, and in each case an appropriate design should be selected.

OTHER CONSIDERATIONS

It must be kept in mind that stopping sight distance is only one of many design and operational considerations in planning an improvement. There certainly can be trade-offs with other features and the potential influence on accident experience.

One geometric consideration that has not been fully investigated relates to sight distance over the concrete barrier in a sag vertical curve in combination with a horizontal curve. There are instances where the driver is afforded safe stopping sight distance, depending on the combination and coordination of horizontal and vertical curvature.

In the areas of operations and safety, there are several matters that should be explored. The first of these relates to the narrowing of the left shoulder on the median side of the freeway on the outside of the curve. This has several potential safety ramifications. One is perhaps an inadequate breakdown area; the second is associated with the reduced lateral clearance to the barrier on the outside of the curve. There may be an optimum dimension that could provide stopping sight distance and not sacrifice other safety features.

It is also appropriate to consider the reduction of the right shoulder on a ramp and the increase in left shoulder width for a curve to the left. A breakdown (disabled vehicle) normally moves to the right shoulder on a ramp. In the case cited above, the left shoulder would provide refuge. Nevertheless, it is the total ramp width (parapet to parapet) that is important in allowing a vehicle to pass a stalled vehicle. Research would be helpful in resolving this issue.

SUMMARY

The highway designer must be made aware of the potential sight distance restrictions on curves of freeways and ramps where a barrier or parapet is present. For many existing situations and for new configurations currently being designed, the solution may be simple. Additional alternatives could be

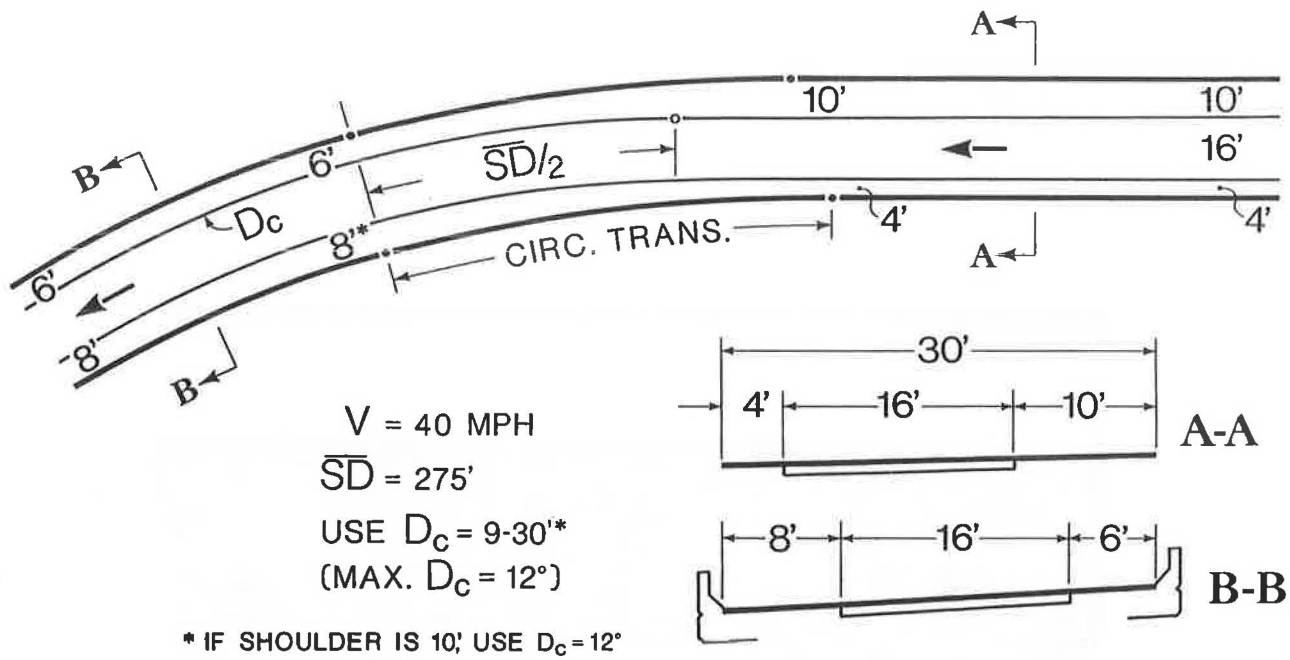


FIGURE 5 An example of provision for sight distance on a ramp with a barrier and a curve left.

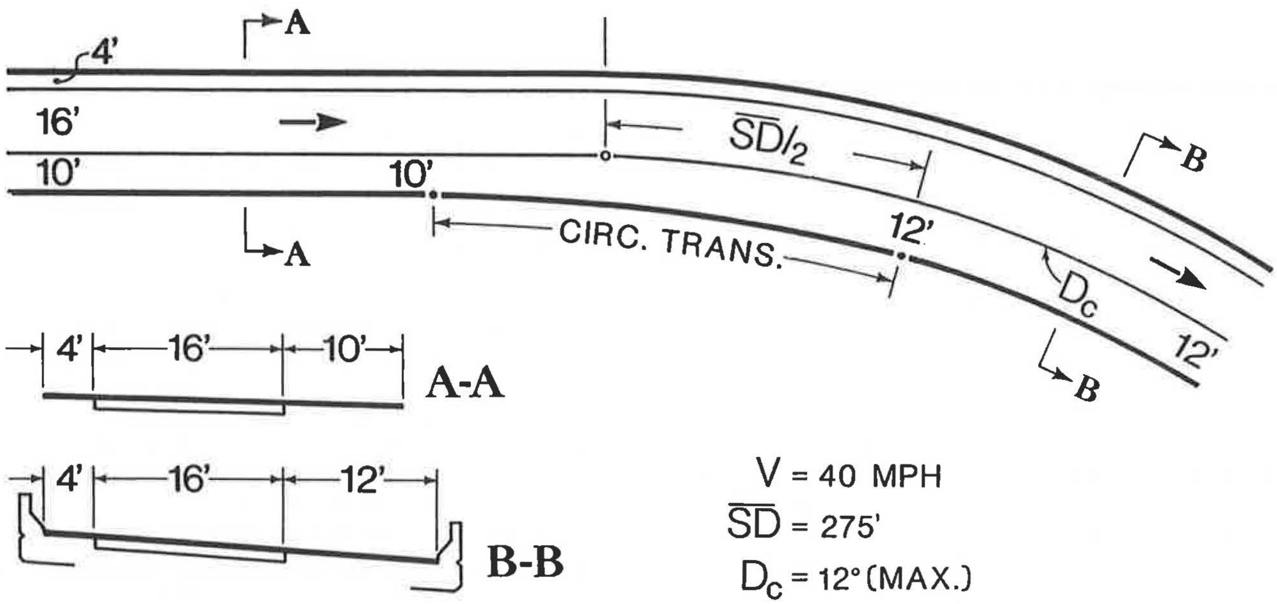


FIGURE 6 An example of provision for sight distance on a ramp with a barrier and a curve right.

developed and analyzed for specific site conditions. Some example solutions for the freeway proper and freeway ramps are presented here for consideration. Other solutions should be documented, based on experience, to assist the engineer in achieving the most cost-effective design for a given set of conditions. Additional research may be required to provide the designer with adequate information concerning trade-offs in design and operational criteria to assure that the optimum design is being achieved.

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Reevaluation of the Usefulness and Application of Decision Sight Distance

HUGH W. MCGEE

One of the most important elements of highway geometric design is sight distance. In *A Policy on Geometric Design of Highways and Streets*, the American Association of State Highway and Transportation Officials has adopted a new sight distance standard known as decision sight distance (DSD). These sight distances are considerably longer than stopping sight distance, giving motorists additional margin for error and sufficient length to maneuver their vehicles at the same or reduced speed rather than to just stop. Nevertheless, there has been some concern that states have not adopted and implemented this standard. To determine if this is true, a limited survey of 15 states was made. A questionnaire was used to determine if the state has adopted the standard—and if it has not, why. Comments were solicited on how the standard should be modified. This paper also critiques a proposed revised AASHTO standard for DSD and concludes with the author's recommendation for a change to the DSD standard.

One of the most important elements for highway geometric design is sight distance. Providing maximum sight line within the vision capabilities of the driver is a desirable goal. If the driver can see what is unfolding far enough ahead, he can handle almost any situation.

Until the issuance of the current AASHTO geometric design manual, there were standards for stopping sight distance, intersection sight distance, passing sight distance, and railroad-highway grade crossing sight distance. Although these sight distance standards have brought about reasonably good design practice for a majority of our roadway system, it was felt that certain situations required longer sight distances. In particular, stopping sight distance, the design criterion that requires minimum sight distance at all points along the road, was thought to be inadequate for situations with high decision complexity, when the development of a potentially hazardous situation is difficult to perceive, and when severe braking is inappropriate. At locations where longer distances are needed, a review of human factors and traffic operations considerations shows that sight distance criteria should be based on the driver's ability to properly react to impending danger. With this concern in mind, the concept of decision sight distance (DSD) was formulated and eventually found its way into the 1984 AASHTO design manual—*A Policy on Geometric Design of Highways and Streets* (*1*)—known as the Green Book.

In that policy, DSD is defined as the distance required for a driver to detect an unexpected or otherwise difficult-to-perceive information source or hazard in a roadway environment that may be visually cluttered, recognize the hazard or its threat potential, select an appropriate speed and path, and initiate and complete the required maneuver safely and efficiently. This definition was developed by Alexander and Lunenfeld and was a key element of the concept of positive guidance (*2*).

At this point a little history of the development of DSD is in order. Although the term "decision sight distance" was first coined by Alexander and Lunenfeld (circa 1975), this longer sight distance concept has its roots with researchers such as the late Donald Gordon of the Federal Highway Administration and Richard Michaels, formerly with the then Bureau of Public Roads.

In his *Dynamic Design for Safety* (*3*), Leisch, drawing on the principles of perceptual anticipation discussed by Gordon (*4*), argues the need for what he labeled "anticipatory sight distance." This distance would provide sight distance at all points along the road adequate for the driver to anticipate changes in design features, intersections, entrances, exits, or trouble spots ahead in sufficient time to take the appropriate action and carry on normally. Using judgment and relationships to "focusing distance," Leisch suggested the following values for anticipatory sight distance:

	Design Speed (mph)					
	30	*40*	*50*	*60*	*70*	*80*
Minimum anticipatory sight distance, ft	600	800	1,100	1,500	2,000	3,000

These anticipatory sight distances were to be measured from the height of eye to road surface. Leisch further suggested that these distances be provided at points of decision or potential hazard, such as approaches to interchanges, at-grade intersections, toll plazas, tunnel portals, road narrowings, lane drops, design speed reduction zones, and the like.

In the article "New Safety and Service Guides for Sight Distance" (*5*), Pfefer discusses anticipatory sight distance, but he also includes "perception sight distance." This notion was based on the first perception of an object in the visual field at which the driver perceives movement (angular velocity). The values suggested were as follows:

	Design Speed (mph)					
	30	*40*	*50*	*60*	*70*	*80*
Perception sight distance, ft	675	775	875	950	1,025	1,100

These values, which are considerably lower than anticipatory sight distance at the higher speeds, were to be provided

Bellomo-McGee, Inc., 8330 Boone Blvd., Suite 700, Vienna, Va. 22180.

TABLE 1 DECISION SIGHT DISTANCE (*1*)

Design Speed (mph)	Time(s)					
	Premaneuver			Decision Sight Distance (ft)		
	Detection & Recognition	Decision & Response Initiation	Maneuver (Lane Change)	Summation	Computed	Rounded for Design
30	1.5-3.0	4.2-6.5	4.5	10.2-14.0	449- 616	450- 625
40	1.5-3.0	4.2-6.5	4.5	10.2-14.0	598- 821	600- 825
50	1.5-3.0	4.2-6.5	4.5	10.2-14.0	748-1,027	750-1,025
60	2.0-3.0	4.7-7.0	4.5	11.2-14.5	986-1,276	1,000-1,275
70	2.0-3.0	4.7-7.0	4.0	10.7-14.0	1,098-1,437	1,100-1,450

continuously along the roadway and measured from the driver's eye to the pavement.

In 1978, analytical and field research was conducted that was documented in the report *Decision Sight Distance for Highway Design and Traffic Control Requirements* (*6*). The DSD values were analytically developed, based on a sequential hazard avoidance behavior model. It was assumed that each of the several steps target detection, perception, decisionmaking, reaction, and maneuver—was performed serially with no time-sharing. Values were established for each of the information processing elements and added to arrive at the total time for DSD. These values were then slightly modified based on results of limited field studies in which drivers were exposed to geometric changes such as lane drops and complicated intersections. The final recommended values were adopted and included in the 1984 AASHTO Green Book (*1*). Table III-3 in the Green Book is shown as Table 1.

STATE SURVEY ON ACCEPTANCE OF DSD

In order to determine to what extent the states have adopted DSD as a design element and if they are using the recommended values as shown in Table 1, a limited survey of a few states was conducted in late 1988. Specifically, the questionnaire shown as Figure 1 was sent to 15 states; of those, 12 replied. The responses are discussed below.

Has Your State Adopted DSD?

Of the 12 states that responded, half indicated that they have adopted DSD and the other half said that they had not. A 100 percent adoption would not be expected because the AASHTO policy manual was released in 1984, and all the states probably have not yet revised their design manuals to reflect any changes or additions in the AASHTO manual. Still, only a 50 percent acceptance of this design criterion indicates that there is no across-the-board acceptance of the values, if not the concept.

If Yes, Indicate How It Has Been Included

Most of the states that have adopted the design criterion have merely referred to the AASHTO manual, or they have duplicated or paraphrased the relevant section dealing with DSD. The State of Maryland has a table for various sight distances, and for DSD requires the following:

	Design Speed (mph)							
	10	*20*	*30*	*40*	*50*	*60*	*70*	*80*
Decision sight distance, ft	225	425	625	825	1,025	1,300	1,625	1,975

For each design speed Maryland has selected the higher values from AASHTO, and for design speeds of 60 mph and higher even longer distances than those in AASHTO have been recommended to allow for a stop maneuver.

If No, Which of the Reasons Apply?

Two states indicated that they are considering adoption of DSD but have not yet formally adopted or rejected it. Three states responded that they have not adopted DSD because the costs of the longer distances required have not been justified. One state responded that new alignments are rare, and it is too costly to provide DSD for rehabilitation projects. Four states said that the guidelines for use of DSD were too vague.

Are DSD Values Too Short, Too Long, or About Right?

The numerical responses to this question were too long, 4; too short, 2; and about right, 7. One state, commenting on its "too long" response, said, "It is difficult to obtain DSD values in urban areas especially in rolling terrain; it is more practical to use stopping sight distance for urban intersections."

Comments

There were several comments that qualified the use or nonuse of DSD:

- Although not applied yet, the concept is workable. What are other states doing?
- We don't use it as often as we should.
- DSD is good concept, but impractical given our budgets and backlog of work.
- DSD is used for placement of warning signs.

1. Has your State adopted Decision Sight Distance (as it appears in AASHTO's Policy on Geometric Design of Highway & Streets) as a design element in your design manual or standard? Yes _______ No _______

2. If YES, please indicate how it has been included or provided appropriate excerpts of your manual. ____

____

____

____

3. If NO, which of the reasons apply:

_______ Under consideration, but have not yet integrated into our manual.

_______ The longer distances required have not been cost justified.

_______ The guidelines for application of DSD are too vague.

_______ Other, ______________________________________

____

____

4. Do you feel the decision sight distance values are:

1) Too long _______

2) Too Short _______

3) About Right _____

If, 1) or 2) please explain ______________________________________

____

____

5. Do you have any comments concerning Decision Sight Distance and its applicability for highway design (e.g., when or where should it be applied)?

____

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____

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Please return to:
Dr. Hugh W. McGee, P.E.
Bellomo-McGee, Inc.
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Vienna, VA 22180

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FIGURE 1 Questionnaire on decision sight distance sent to 15 states.

- DSD should be a routine consideration in all highway design.
- DSD should be applied only at very specific decision points such as at at-grade intersections and complex interchanges.
- Specific application areas of merging, lane drops, ramp exits, and approaches to intersections would be more applicable.
- There are too many variables and specific conditions that are site-specific to effectively utilize this set of criteria.

Several states commented on the object height, which is set at 6 in in the AASHTO standard. One state suggested a higher object height, specifically 4.25 ft, since most of the targets would be other vehicles. A higher object height would essentially result in the allowance of a much less restrictive vertical alignment, even though the distance values remain the same. Another state also said that another vehicle in the lane was the appropriate object to be seen but believed that the appropriate height should be 18 in, reflecting the taillight height.

AASHTO REVISED DSD VALUES

Revisions to the current AASHTO Green Book are being formulated by appropriate committees, and changes to the DSD values is one of them. Table 2 shows the proposed revised DSD values. They have not yet been adopted as final.

As can be seen in Table 2, the recommended revised DSD values are based on the road type and maneuver. The road types are rural, urban, and suburban, and the maneuvers are either to stop or to change speed, path, or direction. A review of the values shows that DSD values are the longest for the urban road for all speeds. This results from the assumption that urban situations are more complex and, therefore, require more time for information processing. While this may be, it can also be argued that in urban situations drivers are more alert, which would result in lower detection time (drivers searching for potential hazards) and lower reaction time.

These values can also be criticized from a practical, cost-effectiveness basis. Adherence to these values requires the design agency to provide the longest sight distances in urban areas, where they are least likely to be realized because of limited right-of-way. Given the objections raised by some states from the survey, it is unlikely that the states would embrace these recommended values.

RECOMMENDED REVISED DECISION SIGHT DISTANCE VALUES

I would like to offer for consideration yet another set of DSD values. These are shown in Table 3. In developing these values, several factors were considered:

- A consistent complaint from the states was that the application guidelines were too vague. Hence, the values are now established for specific situations—interchange exits (left and right); lane drops, lane closures, and merges (all essentially require a lane change); lane shift; and intersections.
- Because the lane shift situation is the least demanding, it requires the shortest sight distance. Sight distance should be measured to the beginning of the shift.
- For intersections, DSD is necessary to be able to see and respond to turn lanes. Therefore, DSD should consider the need for a lane change and be measured to the turn lane itself.
- Lane drops, lane closures, and merges all require a lane change. DSD should be measured to the taper area.
- DSD should be provided at all interchange exits. Longer distances are recommended for left-side exits because of the nonexpectancy factor and because drivers wanting to exit may be at least two lanes removed. Although it could be argued that there are some differences in the time for information detection, processing, and reaction, these differences are not deemed long enough to warrant DSD values for each area.
- Unlike the AASHTO revised values, there is no difference for the type of road or area, for example, rural versus suburban versus urban.
- Only one value is given for each design speed and situ-

TABLE 2 PROPOSED REVISED DECISION SIGHT DISTANCE

DESIGN SPEED (MPH)	Decision Sight Distance Required For Maneuver (Feet)				
	A	B	C	D	E
30	220	500	450	550	625
40	345	725	600	725	825
50	500	975	750	900	1025
60	680	1300	1000	1150	1275
70	900	1525	1100	1300	1450

A: STOP REQUIRED ON RURAL ROAD
B: STOP REQUIRED ON URBAN ROAD
C: SPEED/PATH/DIRECTION CHANGE ON RURAL ROAD
D: SPEED/PATH/DIRECTION CHANGE ON SUBURBAN ROAD
E: SPEED/PATH/DIRECTION CHANGE ON URBAN ROAD

TABLE 3 RECOMMENDED DECISION SIGHT DISTANCE

Design Speed (MPH)	SITUATIONS				
	Interchange 1/ Right Exit	Interchange 1/ Left Exit	Lane Drop/ Closure/ Merge 2/	Lane Shift 3/	Inter-sections 4/
30	N/A	N/A	450	250	450
40	600	825	600	350	600
50	750	1025	750	425	750
60	1000	1275	1000	600	1000
70	1100	1450	1150	725	1150

1/ Sight Distance to Gore
2/ Sight Distance to Taper Area
3/ Sight Distance to Begin of Shift
4/ Sight Distance to Turn Lane

ation. These values should be considered minimums that could be exceeded within cost limitations. There is no reason to have a range as there is with the current design standard.

With regard to the object height, because the purpose of DSD is to provide the motorist with sufficient sight distance of the design feature, the appropriate height should be the pavement surface, that is, 0 ft. Nevertheless, because using the pavement surface as the object height would have the significant effect of increasing the radius of horizontal and vertical curves, using the 6-in object height may be more appropriate. If DSD is being used for design and placement of signs, then a much higher object height can be used.

These values were developed without the benefit of extensive analysis and evaluation and, therefore, are subject to justifiable criticism and review. Nonetheless, regardless of what values are finally selected for inclusion in the AASHTO geometric design policy, certain principles should prevail:

1. Sight distances longer than stopping sight distance are needed for certain situations. They should be identified and specific values should be provided as a standard.
2. DSD values should consider cost implications, especially in urban situations. They should not be unnecessarily long.

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