Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/TX-99/1811-1		

4. Title and Subtitle	5. Report Date
EVALUATION AND MODIFICATION OF SIGHT DISTANCE CRITERIA USED BY TxDOT	September 1998
	6. Performing Organization Code
7. Author(s) Mark D. Wooldridge, Angelia H. Parham, Kay Fitzpatrick, R. Lewis Nowlin, and Robert E. Brydia	8. Performing Organization Report No. Research Report 1811-1
9. Performing Organization Name and Address Texas Transportation Institute The Texas A&M University System College Station, Texas 77843-3135	10. Work Unit No. (TRAIS) 11. Contract or Grant No. Project No. 0-1811
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Transfer Office P.O. Box 5080 Austin, Texas 78763-5080	13. Type of Report and Period Covered Research: September 1997 - August 1998 14. Sponsoring Agency Code

15. Supplementary Notes

Research performed in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

Research Project Title: Develop and Evaluate Sight Distance Criteria at Intersections, Interchanges, and Ramps

16. Abstract

Sight distance is an important consideration in roadway design, affecting many aspects of highway safety and operations. Ramp, interchange, and intersection designs are typically completed in tightly constrained spaces with many structural, earthwork, and roadway features present that may obstruct sight distance. These features are not easily moved; if consideration of sight distance constraints is not given early in the design process, designs may be compromised and a reduced level of safety may be encountered by the public on the completed roadway. After conducting a literature review of design criteria, three case studies of interchange ramps, and a thorough review of the TxDOT *Design Division Operations and Procedures Manual*, recommended revisions were prepared for the manual. These revisions include material intended to clarify and extend the consideration of sight distance in roadway design.

17. Key Words	18. Distribution Statement
Ramp Design, Sight Distance Criteria, Stopping Sight Distance, Decision Sight Distance, Intersection Sight Distance	No restrictions. This document is available to the public through NTIS: National Technical Information Service 5285 Port Royal Road Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	74	

Form DOT F 1700.7 (8-72)

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Report 1811-1
Project Number 0-1811
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Sponsored by the
Texas Department of Transportation
In Cooperation with
U.S. Department of Transportation
Federal Highway Administration

September 1998

TEXAS TRANSPORTATION INSTITUTE
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ACKNOWLEDGMENT

The project team recognizes Robert B. Stone, project director; Alvin Krejci, Jr., Program Coordinator; and Project Advisors Wesley M. Burford, Robert E. Leahey, and Gus Shanine for their time in providing direction and comments for this study. This study was performed in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

The authors would also like to recognize the following persons for helping with data collection, data analysis, and report preparation efforts: Abishai Polus, Molly Marshall, Shirley Kalinec, Jon Collins, and Kollan Pillay.

TABLE OF CONTENTS

TABLE OF CONTENTS

CHAPTER 1
INTRODUCTION

Providing adequate sight distance on a roadway is one of the central tasks of the designer. Adequate sight distance provides motorists with the opportunity to avoid obstacles on the roadway, to merge smoothly with other traffic, and to traverse intersections safely. Ramp, interchange, and intersection designs are typically completed in tightly constrained spaces with many structural, earthwork, and roadway elements present that may obstruct sight distance. These elements are not easily moved; if consideration to sight distance constraints is not given early in the design process, designs may be compromised and may reduce the level of safety on the completed roadway. Sight distance criteria must be presented in a clear, comprehensive, and unambiguous manner to facilitate the completion of satisfactory roadway designs.

A literature review was first completed to review the development of relevant sight distance criteria. Understanding why various criteria were developed and implemented provided a background necessary for the clear understanding of various sight distance equations and recommendations. The review of actual field locations with poor sight distance problems provided a necessary understanding of challenges encountered in design. Three case studies were completed in the project, examining available sight distance at three different sites. Finally, material currently in TxDOT's *Highway Design Division Operations and Procedures Manual*[1] (herein referred to as the *Design Manual*) was reviewed and modifications recommended.

The objectives of this project were to evaluate the sight distance guidelines contained in the *Design Manual* and improve or modify those guidelines where necessary. An emphasis was placed on ramp design, although other sight distance criteria were also evaluated and recommended for modification.

This report provides a review of stopping sight distance, intersection sight distance, decision sight distance, and ramp merge sight distance. Recommended changes to the *Design Manual* centered around updating design values, including additional references to sight distance, and providing additional design tools to help review available sight distance during the design process.

This report is divided into four chapters. Chapter 1 provides background material for the research. The literature review is presented in Chapter 2. Findings from the three case studies are included in Chapter 3, and Chapter 4 presents the recommended changes to the *Design Manual.*

CHAPTER 2
LITERATURE REVIEW

The review of sight distance criteria in the literature focused around three sight distance requirements that frequently apply to various situations encountered in design:

- Stopping sight distance;
- Decision sight distance; and
- Intersection sight distance.

In addition, a fourth category was investigated: ramp merge sight distance. Only a limited amount of literature was available regarding this final topic.

STOPPING SIGHT DISTANCE

According to the American Association of State Highways and Transportation Officials (AASHTO) *A Policy on Geometric Design of Highways and Streets*[2] (herein referred to as the *Green Book*), sight distance is the length of roadway ahead that is visible to the driver. The *Green Book* also states that the minimum sight distance at any point on the roadway should be long enough to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. Although greater length is desirable, sight distance at every point along the highway should be at least that required for a below average driver or vehicle to stop in this distance. The National Cooperative Research Program (NCHRP) recently sponsored a study on stopping sight distance.[3] Most of the following material was obtained from that project's reports.

AASHTO Stopping Sight Distance Model Equations

Stopping sight distances are calculated using basic principles of physics and the relationships between various design parameters. The 1994 *Green Book* defines stopping sight distance as the sum of two components: brake reaction distance (distance traveled from the instant the driver detects an object to the instant the brakes are applied) and the braking distance (distance traveled from the instant the driver applies the brakes to when the vehicle decelerates to a stop).[2] Minimum and desirable stopping sight distances are calculated with the following equation:

$$SSD = BrakeReactionDistance + BrakingDistance \tag{2-1}$$

or more specifically,

$$SSD = 0.278Vt + \frac{V^2}{254f} \tag{2-2}$$

where: SSD = stopping sight distance, m;

V = design or initial speed, km/h;

t = driver perception-reaction time, s; and

f = friction between the tires and the pavement surface.

Roadway grade also affects stopping sight distance, i.e., stopping distances decrease on upgrades and increase on downgrades. SSD for upgrades and downgrades is calculated with the following equation[2]:

$$SSD = 0.278Vt + \frac{V^2}{254\ (f \pm g)} \tag{2-3}$$

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

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

The minimum length of vertical curves is controlled by the required stopping sight distance, driver eye height, and object height. This required length of curve is such that, at a minimum, the stopping sight distance calculated by equation 2-3 is available at all points along the curve. The following formulas are used to determine the required length of crest and sag vertical curves:[2]

For crest vertical curves:

$$L = \frac{AS^2}{200(\sqrt{h_1} + \sqrt{h_2})^2} \qquad when\ S < L \tag{2-4}$$

and

$$L = 2S - \frac{200(\sqrt{h_1} + \sqrt{h_2})^2}{A} \qquad when\ S < L \tag{2-5}$$

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

For sag vertical curves:

$$L = \frac{AS^2}{2(h_3 + S\tan\theta)} \qquad when\ S < L \tag{2-6}$$

and

$$L = 2S - \frac{2(h_3 + S\tan\theta)}{A} \qquad when\ S > L \tag{2-7}$$

where: h_3 = height of vehicle headlights above the roadway surface (m); and
θ = upper divergence angle of headlight beam (most countries use 1 deg; some countries use 0 deg).

The curvature of a crest and sag vertical curve is often characterized by the K-factor, defined as the length of the vertical curve to effect a 1.0 m difference in grade, i.e., the length of vertical curve divided by its algebraic difference in grade. The following equation expresses K[(2)]:

$$K = \frac{L}{A} \tag{2-8}$$

where: L = length of vertical curve, m; and

A = algebraic difference in grade.

Where an object off the pavement such as a bridge pier, bridge railing, median barrier, building, cut slope, or natural growth restricts sight distance, the required offset to that obstacle is determined by the stopping sight distance. When stopping sight distance is less than the length of the horizontal curve, the middle ordinate is determined from the following equation:(2)

$$m = R[(1 - Cos\frac{28.65S}{R})] \qquad when\ S < L \tag{2-9}$$

where: m = middle ordinate, m;

L = length of curve, m;

S = stopping sight distance, m; and

R = radius, m.

When stopping sight distance is greater than the length of the horizontal curve, the following equation can be used:(4)

$$m = R\tan(\frac{I^* - I}{2})\sin(\frac{I}{2}) + R(1 - Cos(\frac{I}{2})) \qquad where\ S > L,\ \ I^* > I \tag{2-10}$$

where: m = middle ordinate, m;

I = length of curve, m;

R = radius, m;

I = deflection angle, deg; and

I* = deflection angle as shown in Figure 2-1, deg.

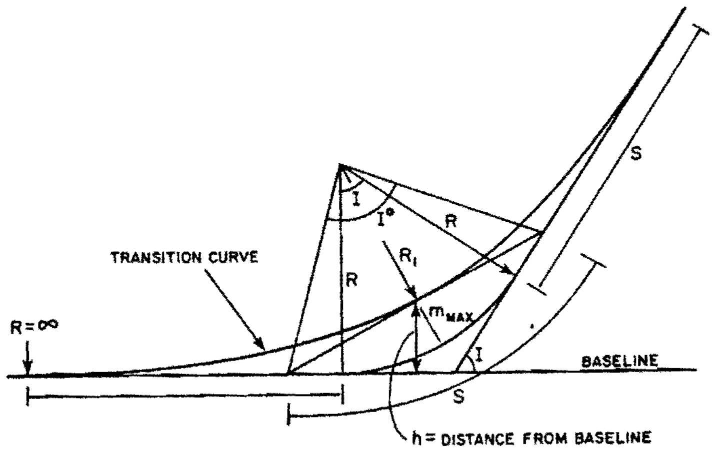


Figure 2-1. Transition Curve for Lateral Clearance[4]

Equation 2-10 may also be approximated as:

$$m = \frac{L(2S-L)}{8R} \qquad where\ S > L \tag{2-11}$$

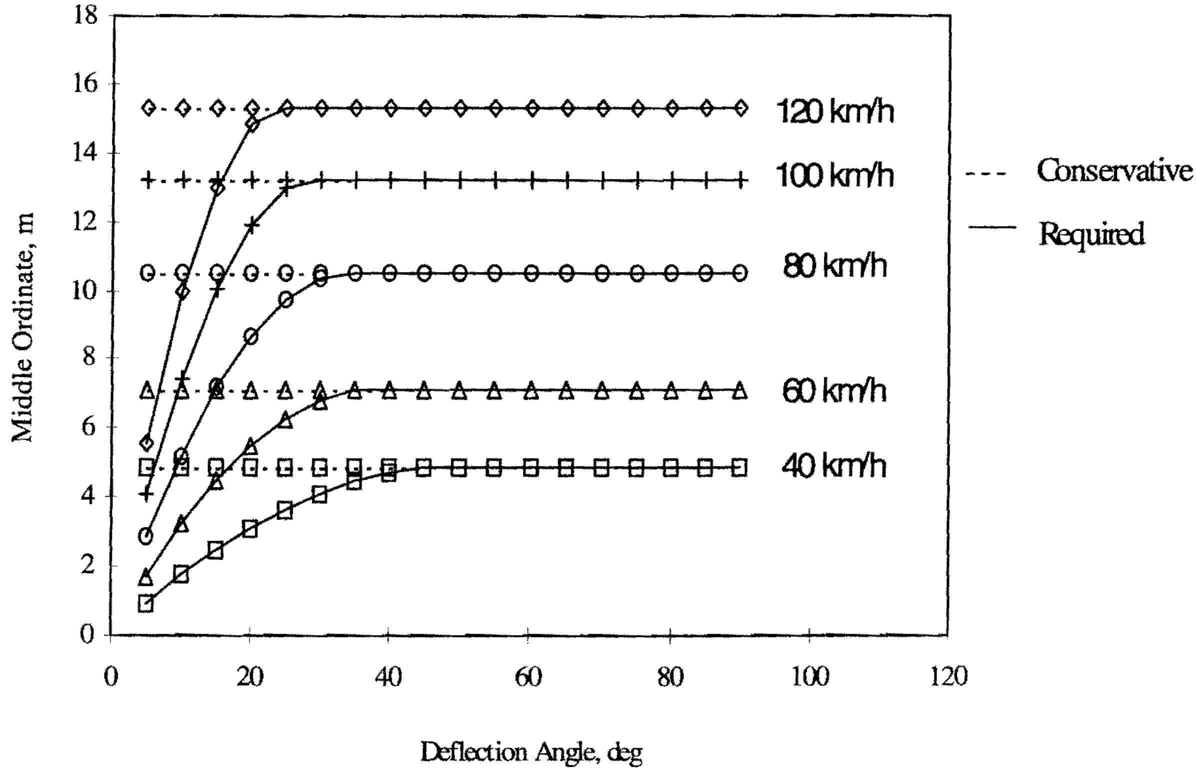
where: m = middle ordinate, m;

L = length of curve, m;

S = stopping sight distance, m; and

R = radius, m.

This equation conservatively approximates the required offset for sight distance obstructions on horizontal curves. Figure 2-2 shows how the use of equations for stopping sight distance less than the length of curve conditions can overstate offset requirements where stopping sight distance is actually greater than length of curve.



Note: Minimum Radius Used for All Design Speeds

Figure 2-2. Middle Ordinate Requirements for S > L

Historical Development of Stopping Sight Distance

Even though the basic stopping sight distance model has remained the same for the past 50 years, the American Association of State Highway Officials (AASHO) and AASHTO publications have addressed several changes in design parameter values within the model during that time. Engineering textbooks addressed the fundamental principles of highway design as early as 1921; however, it was not until 1940 that AASHO published seven highway design documents and formally recognized policies on certain aspects of geometric design. In that same year, these seven documents were reprinted and bound as a single volume entitled *Policies on Geometric Design.*[5]

These policies were revised and amended in a 1954 document, *A Policy on Geometric Design of Rural Highways.*[6] This document was revised and republished under the same title in 1965 and 1972; however, it was called the *Blue Book* because of the color of the cover.[7] The 1994 AASHTO policy[2] and its 1984[8] and 1990[9] predecessors were entitled *A Policy on Geometric Design of*

Highways and Streets, and are commonly called the *Green Book.* The 1994 document is the first AASHTO design policy in metric units. The changes in parameter values in the stopping sight distance model and minimum curve length equations that have occurred from 1940 to the present are summarized in Table 2-1 and discussed in subsequent sections.

Design Speed. The use of design speed in calculating stopping sight distance was first adopted by AASHO in 1940. Design speed was defined as the maximum uniform speed adopted by the faster group of drivers but not necessarily the small percentage of reckless drivers. In 1954, AASHO approximated the assumed speed on wet pavements as a percentage (85 to 95 percent) of the design speed. This reduction was based on the assumption that most drivers will not travel at full design speed when pavements are wet. In 1965, AASHO changed the approximated speed on wet pavements to be a percentage varying from 80 to 93 percent of the design speed. Several researchers have questioned the premise that drivers travel at lower speeds on wet pavements. For example, Knasnabis and Tadi[10] suggested using design speed or an intermediate speed (average of design speed and assumed speed) to compute required stopping sight distances.

AASHO published *A Policy on Design Standards for Stopping Sight Distance*[11] in 1971. This policy introduced a range of design speeds defined by a minimum and a desirable value used for computing stopping sight distance. The minimum value was based on a percentage varying from 80 to 93 percent of the design speed (1965 assumed speeds on wet pavements), and the desirable values were based on the design speed. AASHTO retained the minimum and desirable values in their 1984, 1990, and 1994 policies, but noted that "recent observations show that many operators drive just as fast on wet pavements as they do on dry pavements."[8,9,2]

Perception-Reaction Time. Perception-reaction time is the summation of perception and brake reaction time. Brake reaction time was assumed as 1 sec in 1940;[5] since then, the recommended value for brake reaction time has not changed. In 1940, total perception-reaction time ranged from 2 to 3 sec depending upon design speed. In 1954, the *Blue Book*[6] adopted a policy for a total perception-reaction time of 2.5 sec for all design speeds. The *Blue Book* stated "available references do not justify distinction over the range in design speed." No "available references" were cited; therefore, the reason for this change is not clear.

Design Pavement/Stop Conditions. The basic assumption in calculating braking distances since the 1940s has been a passenger car on a wet pavement with locked-wheel tires throughout the braking maneuver. Wet rather than dry pavement conditions are assumed for design because they

Table 2-1. History of AASHTO Stopping Sight Distance Parameters[3]

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

result in lower coefficient of frictions and longer braking distances. Several researchers have questioned the locked-wheel braking assumption in the literature.

Olson et al.[4] stated that "locked-wheel stopping is not desirable and it should not be portrayed as an appropriate course of action." Their research assumed a controlled stop in which the driver "modulates his braking without losing directional stability and control" and used numerical integration to calculate recommended braking distances. Implicit in such a recommendation is the assumption that drivers can control their vehicle's braking in a stopping situation and avoid locked-wheel braking.

Friction values should be characteristic of variations in vehicle performance, pavement surface condition, and tire condition. Table 2-1 lists the friction factors that were revised according to the prevailing knowledge of the time. Because of the lack of extensive field data, the 1940 AASHO[5] used a 1.25 factor of safety to encompass the variability in assumed friction values. The use of empirical friction factors increased as more studies were completed. Note that friction factors always decreased with an increase in speed. This phenomenon became known as a speed gradient.

Driver Eye Height. Driver eye height values are a combination of driver stature and driver seat height. The design value for driver eye height is selected so that most driver eye heights in current vehicles will be greater than the design value. As shown in Table 2-1, this design parameter has decreased from 4.5 to 3.5 ft over the past 50 years. The change in eye height can be attributed to the increase in the number of small vehicles, changes in vehicle design, and changes in driver seat design. The design eye height was based on the prevailing distribution of drivers and vehicles at the time of each AASHTO publication. The most significant decrease in driver eye height took place between 1954 and 1965, when the eye height changed from 4.5 to 3.75 ft. Although the trend has been a continuing decrease in eye height, most studies now state that the eye height is not expected to decrease significantly in the future.[11,12]

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

Object Height. Over the past 70 years the issue of which object height to use in calculating stopping sight distance has been a much discussed subject. Table 2-1 illustrates the changes in the design object height from 1940 to the present. The object height was set the same as the driver eye height, 5.5 ft in a 1921 highway engineering textbook;[13] however, in 1940 AASHO adopted a 4 in object height as an "average" control value.[5] They stated that "the stationary object may be a vehicle or some high object, but it may be a very low object such as merchandise dropped from a truck or small rocks from side cuts." The surface of the roadway would have provided the safest design, but an object height of 4 in was chosen because large holes in modern pavements were not common and other smaller objects would be easy to avoid.

In 1954, the 4 in object height was justified as "the approximate point of diminishing returns."[6] The use of a zero object height was not justified because of the undue construction costs, and an object height higher than 4 in would exclude lower hazards and produce "dangerously short" lengths of vertical curves. AASHO noted that the connection between object height and vertical curve length displayed a significant relationship: the length of the vertical curve decreased rapidly as the object height increased from 0 to 4 in. Specifically, required lengths of curves decreased by 40 percent when the object height changed from 0 to 4 in but decreased by only 50 to 60 percent when the object height changed from 0 to a height of more than 4 in.

AASHO adopted a 6 in object height in 1965;[7] however, the use of the 6 in object is not well supported in the literature. In fact, the exact paragraph used to justify a 4 in object height in 1954 was also used to justify the 6 in object height in 1965.[6,7] The 1984 and 1990 *Green Books*[8,9] considered a 6 in object height to be "representative of the lowest object that can create a hazardous condition and be perceived as a hazard by a driver in time to stop before reaching it." They also add that the 6 in object is an arbitrary rationalization of possible hazardous objects and a driver's ability to perceive and react to a hazardous situation.

Olson et al.[4] recommended reducing the object height to 4 in, reasoning that increasing the number of small vehicles is causing the average vehicle clearance level to decrease. Olson's rationale was that a 4 in object is less likely to damage or deflect a vehicle than the current 6 in object; therefore, a vehicle is more likely to safely pass over a 4 in object.

Headlight Height and Angle of Divergence. When using headlight sight distance to establish sag vertical curve lengths, a headlight height of 2.0 ft and a 1.0 degree upward divergence of the light beam are generally used for design.[2,8,9] Headlight heights are first mentioned as measuring about 2.0 ft above the pavement surface in the 1940 policy; however, sag vertical curves were not mentioned at that time.[5] In 1954, headlight sight distance appears as one of the design criteria for establishing sag vertical curve length. Length requirements were based on a headlight height of 2.5 ft and a 1.0 degree upward divergence of the light beam.[6] By 1965, the design headlight height had been reduced to 2.0 ft, and it has remained at that value since that time.[7] No documented reason was found for the change from 2.5 to 2.0 ft, but it is consistent with the decreasing of the driver eye height because of decreasing vehicle size during this period.

Middle Ordinate. When designing a horizontal curve, the sight line is a chord of the curve, and the applicable stopping sight distance is measured along the centerline of the inside lane around the curve.[2] The required middle ordinate value—distance from the centerline of the inside lane to the sight distance obstruction—is the criterion most important in providing acceptable stopping sight distance. Calculation of the required middle ordinates for clear sight areas at various degrees of curve is an application of simple geometry that is first mentioned in the 1940 AASHO policy.[5] Although the basic methodology has not changed, the required stopping sight distance has increased due to changes in parameter values with the SSD model. The result of this increase is larger middle ordinate values.

New Model

Despite the criticisms in the literature, most people agree that the AASHTO stopping sight distance model results in well-designed roads, i.e., roads that are safe, efficient, and economical. If so, why initiate a research project to develop a revised model? The need for such a study has been defined elsewhere as follows:[14]

- The current stopping model was based on common sense, engineering judgment, and the laws of physics; however, the parameters within the model are not representative of the driving environment. Thus, the parameters are difficult to justify, validate, and/or defend.
- It has never been established on the basis of data that the provision of longer stopping sight distance results in fewer accidents. Conversely, it has never been established on the basis of data that at least for marginal reductions, provision of shorter stopping sight distance results in more accidents.

As noted, the major criticism of the current model is that its parameters are not representative of the driving environment or safe driving behavior. Thus, although its use results in a good design, it is difficult to justify, validate, and defend as a good model. As a result of these difficulties, a relatively simple driver performance-based model was recommended in a recent NCHRP project[3] as a replacement for the current *Green Book* model.[2] The recommended model is as follows:

$$SSD = 0.278Vt + \frac{0.039V^2}{a} \qquad \textbf{(2-12)}$$

where: SSD = stopping sight distance, m;
V = initial speed, km/h;
t = driver perception-brake reaction time, s; and
a = driver deceleration, m/s^2.

An implicit assumption of a driver performance stopping sight distance model is that the tire/pavement friction must meet or exceed the driver's demands for stopping.

For consistency, it was recommended that the parameters within the recommended stopping sight distance model represent common percentile values from the underlying probability distributions. Specifically, 90th (or 10th) percentile values are recommended for design. The resultant values are as follows:

- One design speed and stopping sight distance;
- Perception—brake reaction time—2.5 s;
- Driver deceleration— 3.4 m/s^2;
- Driver eye height—1080 mm; and
- Object height—600 mm.

The new model results in stopping sight distance, sag vertical curve lengths, and lateral clearances that are between the current minimum and desirable requirements and in crest vertical curve lengths that are shorter than current minimum requirements. (See Figure 2-3 and Table 2-2.)

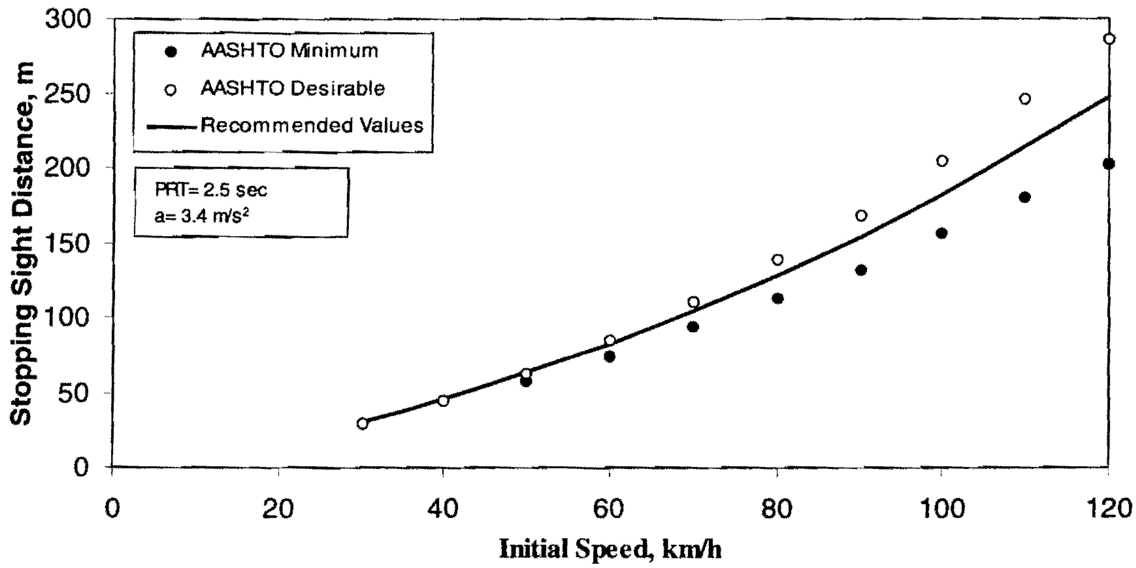


Figure 2-3. Comparison of 1994 AASHTO and Recommended Values for Stopping Sight Distance[3]

Table 2-2. Recommended Stopping Sight Distances for Design[3]

Initial Speed (km/h)	Perception-Brake Reaction		Deceleration (m/s^2)	Braking Distance (m)	Stopping Sight Distance for Design (m)
	Time (s)	Distance (m)			
30	2.5	20.8	3.4	10.2	31.0
40	2.5	27.8	3.4	18.2	45.9
50	2.5	34.7	3.4	28.4	63.1
60	2.5	41.7	3.4	40.8	82.5
70	2.5	48.6	3.4	55.6	104.2
80	2.5	55.6	3.4	72.6	128.2
90	2.5	62.5	3.4	91.9	154.4
100	2.5	69.4	3.4	113.5	182.9
110	2.5	76.4	3.4	137.3	213.7
120	2.5	83.3	3.4	163.4	246.7

Note: Shading represents sight distances that are beyond most driver's visual capabilities for detecting small and/or low contrast objects.

Summary

The determination of required stopping sight distances is based on the distance required to react to a hazard and bring a vehicle to an emergency stop, and, as a minimum, to make that length of roadway visible to the driver. The AASHTO equations were first published in the 1940s and, except for modifications to individual parameters, have not changed since that time. Designs for all types of roadways use these same model and parameter values.

A sensitivity analysis has shown that stopping sight distance is most sensitive to changes in the coefficient of friction; however, assumed coefficients of friction are conservative because they represent wet pavements and worn tires. A question arises related to different coefficients of friction for different types of roadways. Lower volume roads may not be built or maintained to the standards of higher volume roads; however, they may have higher friction values because of lower traffic volumes. Higher volume roads are built and maintained at higher standards but have more traffic to wear down the surface.

Stopping sight distance is also sensitive to changes in perception-reaction time. Some researchers believe that the perception-reaction time should be longer to include all potential situations, while other researchers feel it should be shorter. Some researchers believe that perception-reaction time may vary according to type of roadway. The *Green Book*[2] notes that drivers on urban facilities confronted by possible conflicts with crossing vehicles may be more alert than the same driver on a limited access facility; however, the driver on the lower classification road also may be distracted by adjacent roadside developments, whereas the driver on the limited access facility may be more attentive due to interaction with other traffic.

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

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

A new model for determining stopping sight distance requirements for geometric design of highways was developed in a recent NCHRP study.[3] The new model is based on parameters describing driver and vehicle capabilities that can be validated with field data and defended as safe driving behavior. More than 50 drivers, 3,000 braking maneuvers, 1,000 driver eye heights, and 1,000 accident narratives were used in developing the recommended parameter values for the new model. The recommended values are attainable by most drivers, vehicles, and roadways. The new model results in stopping sight distances, sag vertical curve lengths, and lateral clearances that are between the current minimum and desirable requirements, and crest vertical curve lengths that are shorter than current minimum requirements.

DECISION SIGHT DISTANCE

The concept of decision sight distance (DSD) was first addressed in a 1966 paper by Gordon.[15] In his paper, Gordon talked about the concept of "perceptual anticipation." The concern was that the existing stopping sight distance values were too short for situations that required high-decision complexity.

Building on Gordon's argument, Leisch studied this concept further and defined the term "anticipatory sight distance."[16] This distance provides the necessary time for drivers to anticipate changes in design features (such as intersections, interchanges, lane drops, etc.) or a potential hazard in the roadway and perform the necessary maneuvers. Leisch developed recommended values for anticipatory sight distance (see Table 2-3) using judgment and relationships to "focusing distance." The sight distances in Table 2-3 were to be measured from eye height to road surface.

Table 2-3. Anticipatory Sight Distance Values Recommended by Leisch[16]

Design Speed (km/h)	48	64	81	97	113	129
Minimum Anticipatory Sight Distance (m)	183	244	336	458	610	915

In a 1975 study by Alexander and Lunenfeld,[17] the term "decision sight distance" was defined as follows:

> "...the distance at which drivers can detect a hazard or a signal in a cluttered roadway environment, recognize it or its potential threat, select an appropriate speed and path, and perform the required action safely and efficiently."

Guidelines on DSD values were developed in a 1978 FHWA study by McGee et al.[18] At the time of McGee et al.'s study, AASHTO provided guidelines for only three types of sight distance: stopping, passing, and intersection.[7] Therefore, the objective of the study was to relate the concept of DSD to specific roadway, traffic, and driver characteristics.

Recommended values for DSD were developed based on the hazard-avoidance model. This model was developed and modified in previous research efforts and consists of the following six variables:[19,20,21]

1. Sighting: Baseline time point at which the hazard is within the driver's sight line.
2. Detection: Time for driver's eyes to fixate on the hazard.
3. Recognition: Time for brain to translate image and recognize hazard.
4. Decision: Time for driver to analyze alternative courses and select one.
5. Response: Time for driver to initiate response.
6. Maneuver: Time for driver to accomplish a change in path and/or speed.

The total time required from the moment that the hazard is visible to completion of the maneuver is determined by adding all six variables.

Recommended values for each variable in the hazard-avoidance model were obtained from existing literature and then validated through field studies. The field studies required test subjects to drive through a course and respond to changes in roadway geometrics (primarily lane drops). While the test subjects were driving the course, the times to perform the tasks defined in the hazard-avoidance model were measured.

The results from the study by McGee et al.[18] were used to develop recommended DSD values based on the design speed of the roadway. The recommendations were adopted and introduced in the 1984 AASHTO *Green Book*.[8]

In 1989, a survey was conducted to determine to what extent states had adopted DSD as a design element. Twelve of the 15 states targeted for the survey responded. Survey results revealed that half of the responding states had not adopted DSD for the following reasons:

- Costs for longer distances required were not justified.
- Guidelines for the use of DSD in the 1984 *Green Book*[8] were too vague.

Other comments received from the survey were:

- DSD is a good concept but impractical because of budgets and backlog of work.
- DSD is used for placement of warning signs.
- DSD should be a routine consideration in all highway design.
- DSD should be applied only at very specific decision points such as at-grade intersections and complex interchanges.

Current Guidelines in *Green Book*

Because the initial guidelines presented in the 1984 *Green Book*[8] were very vague and difficult to apply, the guidelines were updated in the 1990 *Green Book*[9] and remained unchanged in the 1994 revision.[2]

In the 1994 *Green Book*,[2] decision sight distance is defined as follows:

> "...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 potential threat, select an appropriate speed and path, and initiate and complete the required safety maneuver safely and efficiently."

The *Green Book*[2] recommends that DSD be provided when drivers must make complex or instantaneous decisions, when information is difficult to perceive, or when unexpected or unusual maneuvers are required. Examples of critical locations where DSD should be considered are:

- Interchange and intersection locations where unusual or unexpected maneuvers are required;
- Changes in cross section such as toll plazas and lane drops; and
- Areas of concentrated demand where there is apt to be "visual noise" whenever sources of information compete, such as those from roadway elements, traffic, traffic control devices, and advertising signs.

Recommended values for DSD are shown in Table 2-4. These values are substantially greater than stopping sight distance because of the additional time allowed to maneuver a vehicle. The recommendations in Table 2-4 are based on the location of the road (urban, suburban, or rural) and the type of maneuver required (change speed, path, or direction). As shown in this table, shorter DSD values are required for rural roads and when a stop maneuver is involved.

INTERSECTION SIGHT DISTANCE

At-grade intersections have long been a focal point for vehicle conflict. Since the first days of geometric design, the crossing of two roadways has necessitated a compromise between mobility and safety. Over time, formal guidelines for establishing clear sight requirements at intersections have evolved. Developed by AASHO (American Association of State Highway Officials) and later AASHTO (American Association of State Highway and Transportation Officials), these guidelines outline the procedures and requirements necessary for the establishment of safe distances that allow vehicles approaching an intersection to either regulate their speeds such that safe passage across the intersection is achieved by both vehicles or to effect regulatory control on the minor roadway by requiring vehicles to stop and proceeding when the major roadway is clear.

The first formal presentation of intersection sight distance (ISD) requirements appeared in the 1940 AASHO publication which is part of the *Policies on Geometric Design* publication.[5] This initial discussion contained procedures for three general classifications of intersections. Over the

Table 2-4. Recommended Decision Sight Distance Values in the 1994 *Green Book*[2]

Design Speed (km/h)	Decision Sight Distance for Avoidance Maneuver (m)[1]				
	A	B	C	D	E
50	75	160	145	160	200
60	95	205	175	205	235
70	125	250	200	240	275
80	155	300	230	275	315
90	185	360	275	320	360
100	225	415	315	365	405
110	265	455	335	390	435
120	305	505	375	415	470

[1] A: Stop on rural road
B: Stop 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

next five decades, subsequent publications furthered the concept of intersection sight distance and refined the clear sight requirements. The 1990 *Green Book*[9] included four cases for ISD procedures; the 1994 *Green Book*[2] added an additional case of vehicles turning left off of the major road onto the minor road for a total of five cases. The classifications and cases are summarized in Table 2-5.

The basis of all *Green Book*[2] intersection sight distance requirements is stopping sight distance which was detailed earlier in this report. A vehicle approaching an intersection has the choice of accelerating, slowing, or stopping, depending on the intersection control. The application of intersection sight distance is discussed with regard to a sight triangle, which is a mechanism for applying sight distance along each leg of an intersection. A sight triangle is simply an unobstructed distance along both roadways and across the included corner for a specified distance which should be kept clear of any sight obstructions. A brief discussion of each case in the 1994 policy follows.

Case I

As presented in the original 1940 policy, the concept of a Case I intersection is to allow drivers to regulate their speeds to achieve safe passage across the intersection for both vehicles. That means that the drivers have to be able to see an approaching vehicle along the other leg of the sight triangle and moderate their speed accordingly. The policy states that the time allotted for this distance is arbitrary, combining the 2 sec for perception-reaction time plus an additional second for a total time of 3 sec. Using the simple relationship of distance equals the product of velocity and time, and substituting 3 sec for the time, the distance traveled could be expressed as:

$$d = 0.278 \cdot 3V = 0.83V \qquad \textbf{(2-13)}$$

where: V = speed, km/h

The various policies since 1940 have shown no revisions to Case I ISD. However, the *Green Book* clearly states that intersections designed with Case I ISD are not necessarily safe and recommends that this case only be used on lightly traveled two-lane roadways where the cost of providing increased sight distance is prohibitive.

Table 2-5. History of AASHTO Intersection Sight Distance Cases

<table>
<tr><td>1940 AASHO Classifications
• Case I - Enabling Vehicles to Begin to Change Speed
• Case II - Enabling Vehicle to Stop
• Case III -Vehicle Stopped at a Preference Road</td><td rowspan="2">1994 AASHTO Cases
• Case I - No Control
• Case II - Yield Control
• Case III - Stop Control on Secondary Road
-- Case IIIA - Crossing Maneuver
-- Case IIIB - Turning Left into a Crossroad
-- Case IIIC - Turning Right into a Crossroad
• Case IV - Signal Control
• Case V - Left Turns from the Major Roadway</td></tr>
<tr><td>1990 AASHTO Cases
• Case I - No Control
• Case II - Yield Control
• Case III - Stop Control on Secondary Roads
-- Case IIIA - Crossing Maneuver
-- Case IIIB - Turning Left into a Crossroad
-- Case IIIC - Turning Right into a Crossroad
• Case IV - Signal Control</td></tr>
</table>

Case II

In contrast to Case I, where the concept is to allow vehicle operators to control their speed, Case II is designed to allow the vehicle on the major road to continue at its current speed without stopping. The vehicle on the minor road should regulate its speed and decelerate to a full stop. In order to come to a complete stop, stopping sight distance must be provided along the minor road. Stopping sight distance is computed according to the formula given in equation 2-14:

$$d = 0.278Vt + \frac{V^2}{254f} \qquad \textbf{(2-14)}$$

where: V = speed, km/h;
t = perception-reaction time, s; and
f = friction factor.

With the perception-reaction time equal to 2 sec and the coefficient of friction equal to 0.40, the equation becomes:

$$d = 0.556V + 0.0098V^2 \qquad \textbf{(2-15)}$$

where: V = speed, km/h.

Although the stopping sight distance values for Case II are tabulated in the AASHTO publications, the proper procedure is to utilize the geometric relationship of similar triangles to determine the required stopping distance along the minor roadway. The procedure is to solve for the distance required along the minor roadway, d_b. When d_b is known, the critical speed on the minor road V_b can be determined mathematically through the use of similar triangles. When this is done, the equation for Case II ISD becomes:

$$d_b = 0.556V_b + 0.0098V_b^2 \qquad \textbf{(2-16)}$$

where: V_b = critical speed at which vehicle b can stop in distance d_b, km/h.

In this formulation, if the critical speed is lower than the design speed of the minor road, the intersection will function as envisioned. If, however, this speed is greater than the design speed of the minor roadway, the minor roadway must be posted with either speed warning signs or a stop sign. The *Green Book* clearly states that Case II intersections become less appropriate as traffic levels increase.

Case III

Case III ISD is applicable when there is a stop sign on the minor road. In this situation, the minor road vehicle must be able to see for a sufficient distance along the major roadway to start moving and clear the intersection. The time required to clear the intersection is dependent on the perception time, the time required to engage the vehicle, and the time required to accelerate across the intersection. The basic formulation for Case III ISD is:

$$d = 0.278V(J + t_a) \qquad \textbf{(2-17)}$$

where: d = sight distance along the major highway from the intersection, m;

J = sum of perception-reaction and gear actuation time, s; and

t_a = time required to accelerate and travel the distance S to clear the major highway pavement, s.

In the 1984 *Green Book*, the Case III policy was revised to accommodate different types of intersection maneuvers that could be performed from a stopped vehicle on the minor roadway. Specifically, Case III was broken into three sub-cases as follows:

- Case IIIA - Crossing Maneuver
- Case IIIB - Turning Left into a Cross Road
- Case IIIC - Turning Right into a Cross Road

The 1984 *Green Book* presented graphical solutions for each of the three sub-cases. In 1990, although the formulations were not changed, the underlying parameter values that led to the graphical solution were revised. The 1994 *Green Book* had no changes in the formulations with the exception of converting them to metric.

Case IV

AASHO publications previous to 1957 did not discuss signalized intersections. The first reference of this situation was acknowledged then dismissed by stating that normal ISD requirements are not necessary at signalized intersections.

The 1984 *Green Book* gives the first discussion of the Case IV condition by stating that due to the operational considerations inherent at an intersection operated by signalized control, Case III sight distances should be available to the driver. The supporting evidence for this argument is that increased hazards at the intersection warrant this distance, particularly in the event of failure of the signal, violations of the signal, or other possibilities such as right turns on red. Neither the 1990 nor 1994 policies furthered the 1984 discussions.

Case V

The 1994 *Green Book* contained the first write-up for vehicles stopped on the major road and turning left onto the minor road. Labeled as Case V, the driver turning left must be able to see a sufficient distance ahead to turn left and clear the opposite lane before a vehicle in that lane reaches the intersection. The required sight distance can be expressed as:

$$d=0.278V(J+t_a) \tag{2-18}$$

where: d = sight distance along the major highway from the intersection, m;

J = sum of perception-reaction and gear actuation time, s;

t_a = time required to accelerate and travel the distance S to clear the major highway pavement, s.

This formulation is equivalent to Case IIIA in all regards.

Alternative Models

Although the AASHTO models have been in existence for over 50 years, many highway engineers have felt that the ISD required for Case IIIB and IIIC may be excessive while the ISD required for Case I may not be large enough. Many alternative methods for determining a safe distance for intersection operations have been proposed and utilized in many states or local agencies. Some of these models utilized variations of the AASHTO equations, such as incorporating more information about deceleration maneuvers, while others were completely new formulations using vehicle parameters.

Throughout the investigation into alternative models, one method that has been around for more than 50 years is gap acceptance. As used for stop controlled intersections, gap is the elapsed time between the arrivals of two major road vehicles at the stop sign. Each gap is an opportunity for a minor road vehicle to leave the minor road and enter the major road. The minor road vehicle has the choice to accept or reject the opportunity. Many researchers have investigated gap acceptance using increasingly sophisticated analyses.

In 1944, Greenshields et al.[22] began the first investigation into the performance and characteristics of traffic at intersections. Limiting the field of study to urban streets, they investigated the starting performance of vehicles at signalized intersections, along with capacity. In addition, the study examined behavior patterns at unsignalized intersections, including both those with stop control and those with no control.

To obtain the data required for the analysis, Greenshields et al. utilized time-lapse photography which allowed researchers to study the behavior of the minor road traffic as it moved into the major road traffic stream. A camera was mounted high over the intersection on a building roof and pointed down at the ground. This technique also required an unobstructed view of all four approaches. Although finding intersections with the requisite building close to the corner and the required view limited the available study sites, it eliminated the need for costly equipment to hang the camera overhead and allowed an operator to be with the camera during the study. This ground-breaking study into gap methodology resulted in the basic definitions of gap acceptance.

If a minor road vehicle entered the intersection between two successive vehicles, it was said to have accepted the time gap. If it did not enter the intersection, it was said to have refused the time gap. Greenshields et al. recorded 416 observations: 103 accepted time gaps and 313 rejected time gaps. According to the results obtained, drivers accepted time frame gaps ranging from 3 to 20 frames with most of the sample concentrated in the 5 to 14 frame area. Greenshields et al. defined an average minimum time gap as the time gap that would be accepted by more than 50 percent of the drivers. The average accepted minimum time gap was 10 frames, which corresponds to a gap acceptance of 6.1 sec.

In 1950, Raff and Hart[23] undertook a study entitled a "A Volume Warrant for Urban Stop Signs" in which the goal was to define and present a numerical warrant for stop sign installation based upon the delaying effect. Due to limited resources, the study was restricted to right-angle isolated urban intersections.

As presented by Raff and Hart, a logical approach to defining a new warrant for stop sign installation is based upon the question of how many side street vehicles would be expected to interfere with the major street vehicles if there were no stop signs. Raff and Hart clearly point out that the decision of what constitutes a substantial proportion of side street vehicles is an arbitrary judgment, but that the underlying premise for this study is 50 percent. To collect the field data necessary for the basis of the warrant, Raff and Hart required the following data:

- major and minor road volumes;
- arrival and departure times of vehicles on both streets; and
- number of delayed cars on the minor road.

Citing several limitations to the camera approach as employed by Greenshields, Raff, and Hart used a pen-based recorder that allowed the tracking of up to 20 events simultaneously. The device was operated by personnel within a parked car near the intersection and used for all data collection. Raff and Hart's studies were conducted at four intersections in New Haven, Connecticut.

In the analysis, Raff and Hart utilized two terms for specific time-internal occurrences that were common in the data. The first term, gap, is defined as the interval from the arrival of one major road vehicle at the intersection to the arrival of the next major road vehicle. This is the same definition that Greenshields used. The second term, lag, is defined as the interval from the arrival of a minor road vehicle to the arrival of the next major road vehicle. By viewing the strip chart output from the graphic time recorder, Raff and Hart were able to determine not only the various gaps and lags associated with the operations at the intersection, but they were also able to record the acceptance and rejection decisions made by the minor road vehicles.

For the analysis, Raff and Hart distinguish quite clearly between gaps and lags. Specifically, Raff and Hart detail the fact that combining gaps and lags into a single analysis is illogical for two reasons. The first reason is based on the premise that a driver can only accept one gap or lag, while they can reject several. Therefore, Raff and Hart point out the fact that if all gaps and lags are counted equally, reporting on the intervals accepted is not a true measure of the percentage of drivers that accepted. If the percentage of intervals accepted is to be representative of the percentage of drivers who accepted a time interval of a specific size, then the same number of intervals must be counted for all vehicles. This can be accomplished only by ignoring the gaps and counting the accepted lags. The second reason for not counting gaps and lags in the same analysis is that a gap and a lag are different quantities since the intersection is clear during the entire duration of the gap, while it is blocked for a short time at the beginning of the lag.

As a result, Raff and Hart define their stop sign warrant based only on lags. Raff and Hart further define a quantity known as the critical lag, L. L is defined as the size lag which has the unique property of having the number of accepted lags shorter than L equal to the number of rejected lags longer than L. The critical lag can be determined by plotting two cumulative distributions on the same chart, the number of accepted lags shorter than a time, t, and the number of rejected lags longer than a time, t. The intersection of the two curves is the critical lag, L.

Raff and Hart note that the critical lag is not the same as the average-minimum time gap quantity which arose from Greenshields' work. At each intersection in Raff and Hart's study, the Greenshields value is roughly 0.2 sec longer than the critical lag. Raff states that the critical lag value is preferable to that of the average-minimum time gap since it is defined in such a way as to relate directly to the operation of the intersection. The objective of both quantities is to simplify the calculation of the number of vehicles delayed by allowing the assumption that all lags shorter than a specified value are rejected, while all lags larger than a specified value are accepted.

Raff, Hart, and Greenshields contributed a great deal of understanding to the principle of gap acceptance. Further studies took these principles and refined not only the methodology but also the definitions associated with gap acceptance. Wagner,[24] for example, defined the types of gaps that can occur on the two-way, two-lane major road. The four types of gaps are labeled as Near-Near, Near-Far, Far-Near, or Far-Far. In the terminology, near refers to the traffic lane next to the minor road. Far refers to the traffic lane on the other side of the major road. A Near-Near gap then is defined as a gap between two successive vehicles in the lane closest to the minor road. A Far-Near gap is defined as a gap between two successive vehicles, the first being in the far lane, and the second being in the near lane. Additionally, the scenarios of Near-Near and Far-Far involve gaps from vehicles traveling in the same direction, whereas Near-Far and Far-Near gaps involve vehicles traveling in opposing directions.

Findings related to gap acceptance from Wagner's 1966 study follow:

- Gaps and lags should be treated separately.
- A gap of a given size is more acceptable than a lag of the same size.
- Drivers accept smaller gaps and lags in peak periods.
- Sequence of gap formation has a significant impact on acceptance during the peak period. Near-Far and Far-Near formations should be treated separately, while Near-Near and Far-Far formations can be grouped.
- The sequence of gap formation had no impact on acceptance during non-peak periods.

Other researchers continued the formulation of gaps and the acceptance decisions that are made. Tsongas and Weiner[25] investigated the probability of gap acceptance between day and night conditions. Based on that study, the following points are listed:

- There were no significant differences in the formation of gaps at day and night. However, as volume increased, longer gaps were present more often at night than during the day.
- No difference exists in the acceptance probabilities of day and night drivers for gaps of medium size. For short gaps and long gaps, statistical analysis indicates that the day/night distribution may not be homogeneous. The authors state that further research will be needed to clarify this point.
- Female drivers take longer gaps.
- Turning goods vehicles (trucks) exhibit significantly longer gap acceptance.

Once the basic methodology for determining gap had been developed, some researchers turned towards the analysis of the data to examine the best method of finding the key numbers for critical gap and the distribution of the gap and lag acceptances.

In 1977, Miller[26] evaluated nine different methods of estimating the parameters of gap acceptance. The various methodologies were each tested using the same set of artificial data. Some evaluations examined only gaps or lags while others analyzed both. The goal of each analysis was to determine the proportion of people who will accept a gap of a given size.

Of the nine methodologies, two examined only lags. The concept of using only lags is based on the fact that each driver at an intersection only experiences one lag with a corresponding decision to accept or reject. There is then a one-to-one correspondence between the drivers and their decisions. The downside of this is that a significant amount of data collected is not utilized for understanding driver behavior. Each of the nine methods has strengths and weaknesses, including: satisfactory results when the traffic volumes are fairly light; good results from probit analysis for the range of traffic volumes only if the class intervals are kept small; skewed results if the distribution of the lags resembles a log-normal distribution; small sample sizes which resulted in biased estimators; undue influence by drivers with a large time differential between their accept and reject times; maximum likelihood gave the same results as a probit analysis if the distribution of gaps/lags are normal; and procedures which used all of the data (gaps and lags) for estimating the parameters gave more accurate results.

The 1985 Highway Capacity Manual (HCM)[27] utilized the concept of a critical gap in the discussion pertaining to unsignalized intersections. As reported in the manual, the critical gap depends on a number of factors, including:

- type of maneuver;
- type of minor road control (STOP/YIELD);
- average running speed on major road;
- number of lanes on major road; and
- geometrics and environmental conditions at the intersection.

As reported in the HCM, values of critical gap range from a minimum of 5.0 sec for a right turn from a yield-controlled intersection to a maximum of 8.5 sec for a left turn at a stop-controlled intersection with four lanes on the major road. Additionally, adjustment factors for curb radius, turning angles, acceleration lanes, volume, and restricted sight distance can be used to increase or decrease the critical gap. The maximum adjustment is +/- 1.0 sec with the maximum critical gap set at 8.5 sec.

In 1988, Uber and Hoffman[28] investigated three different hypotheses of how drivers actually accept a gap. The Time Hypothesis theorizes that each driver has a fixed critical gap or lag measured in time units regardless of the speed of the approaching vehicle. The Modified Time Hypothesis states that the critical gap for each driver is also based on the speed of the approaching vehicles. The Distance Hypothesis states that each driver has a critical distance that is acceptable for gaps/lags regardless of the speed of the approaching vehicles.

The researchers' review of other studies on gap acceptance led to the belief that Greenshields' work in 1947 disproves the Distance Hypothesis. Supplementing the literature review with field study, Uber and Hoffman report the following points:

- Critical gap model is based on both time and approach speed.
- Critical lag is based only on time.
- The capacity of an intersection may increase as speeds increase.

In 1989, Fitzpatrick(29) performed a review of gap acceptance literature which resulted in these findings:

- Drivers accept shorter gaps in darkness.
- The major street speed significantly affects the acceptance of gaps.
- The gaps accepted may be influenced by the type of control (YIELD [*Green Book* Case II] verses STOP [*Green Book* Case III]).

Fitzpatrick concluded that gap acceptance methodology could be used to provide an intersection sight distance policy but that further research on day versus night, urban versus rural, and high volume versus low volume conditions was necessary.

In 1994, Abou-Henaidy, Teply, and Hunt published a summary of gap acceptance investigations in Canada.(30) The focus of their investigations was left-turning vehicles at unsignalized T-intersections. A total of 3,400 left-turning vehicles across five intersections was included in the data set. The data set included 17 pieces of information on each situation, including both driver and vehicle characteristics. Similar to the 1977 Miller study, the researchers investigated a number of different models to analyze the data. The final analysis used a binary logit model to estimate the parameters of the model. A logit model describes the behavior of a single person in selecting an outcome from a set of alternatives. In the case of a binary logit, the choice set is limited to a yes or no situation. In the application to gap acceptance, the choice set is to either accept or reject the available gap.

The most significant variables in the description set were available gap size, gender, the presence of passengers, and the front queue delay. Speed and type of traffic on the main line roadway were also significant as well as the presence of vehicles in a queue behind the front vehicle. An interesting result was found with result to the front delay analysis. The researchers found that vehicles in motion at the front of the queue tended to accept short gaps and stay in motion. Once drivers were stopped, however, they tended to wait for longer gaps. After approximately 30 sec of waiting, this tendency reversed itself and drivers again began accepting increasingly shorter gaps.

This research only addressed left-turning vehicles at T-intersections. Other situations such as right-turning vehicles and the more complicated four-way intersections were not considered. Much work has been done to model the characteristics of gap acceptance in order to create equations or nomographs for standardized use. Madanat et al.[31] examined gap acceptance criteria as a logit formulation. Several models for gap acceptance were proposed and tested which utilized variables such as the length of the gap, the number of gaps rejected and various measures of delay. Based on the result of their modeling, the authors chose a model based solely on the length of the gap for further analysis. This model was then inserted into a queuing model to estimate delay parameters. Although this work was calibrated for only 30 min of right-turn data, the results show closer agreement than Raff's method.

Lerner et al.[32] also studied several issues associated with gaps in a 1995 study. Gap acceptance behavior was examined for through-, right-, and left-turning maneuvers. The results of this study indicate a 7 sec 50th percentile gap acceptance point across all age groups and driving conditions. The 85th percentile point is approximately 11 sec. Neither the maneuver (left, right, or through), driving conditions (day or night), nor site was observed to significantly affect the gap selected. In daytime, males were observed to accept gaps that were shorter by approximately 1 sec than those accepted by females. Age differences were apparent during the day but not at night.

The results of the study indicated minimal differences for driving conditions (day or night) and site effects across all ages. The group requiring the longest times (i.e., the highest rejected lags) were the older drivers. The influence of the maneuver type was very pronounced in this group, with the right-turn movement requiring approximately 1.4 sec longer than the left turn and approximately 1.7 sec more than the through movement.

In an attempt to answer many of the questions concerning intersection sight distance, such as the appropriate methodologies and parameter values, the National Cooperative Highway Research

Program (NCHRP) funded a study on ISD which concluded in 1996. The explicit goal of this project was to examine the current AASHTO methodology and recommend new or revised models and/or parameters for Cases I through V. Published in 1996, the final report[33] made the following recommendations:

- Case 1 rationale was changed to allow both vehicles the opportunity to stop rather than adjust speed, because adjusting speed requires both drivers to take the correct action. A new model was formulated which accounts for this change.
- Case II ISD values were lengthened over current AASHTO values as a result of affording a driver approaching a yield-controlled intersection greater flexibility over a stop-controlled intersection.
- Case III methodology was changed to gap acceptance. The standard length of the departure sight triangle along the major road should be 7.5 sec for passenger car vehicles, 9.5 sec for single unit trucks, and 11.5 sec for combination trucks. The length of the departure sight triangle on the minor road should be 4.4 m.
- Case IV sight distance follows the new values and methodology for Case III utilizing gap acceptance. When signals are to be placed on flashing operation for low-volume periods, the departure sight triangles for Case III operations should be provided. Where right-turn-on-red operations are allowed, Case III departure sight triangle for right turns must be provided.
- Case V operations are modeled on gap acceptance operations and include adjustments for the number of lanes to be crossed.

RAMP MERGE SIGHT DISTANCE

In a 1960 study, Pinnell[34] reviewed entrance ramp characteristics. One area investigated was that of sight distance. According to Pinnell, drivers exhibit a more desirable entrance ramp behavior when provided adequate sight distance to the main lanes. If drivers were provided with a view of the main-lane vehicles from 200 ft upstream of the ramp nose, the drivers were able to merge with low relative speed differentials between their vehicles and the vehicles on the main lanes. This view allows the drivers to adjust their speed to provide entry to a suitable gap in the traffic.

Another examination of sight distance required on an entrance ramp was completed by Bhise[35] in 1973. Although the study was oriented towards vehicle design and the constraints placed on drivers by roof pillars and other vehicle obstructions, the performance of the test drivers was monitored on a wide variety of ramps. In this study, the search and scan behavior of drivers on on-ramps was monitored with an eye-mark camera system. Drivers on the ramps were observed to be actively searching for main-lane vehicles as much as 10 sec prior to the ramp nose. A recommendation was made that drivers be able to observe traffic on the main lanes for 10 sec prior to and after the nose of the ramp. This study was very limited in scope, however, and the findings were based on data from four young male drivers only.

CHAPTER 3
RAMP CASE STUDIES

Sight distance restrictions in freeway interchanges can be problematic because of the likelihood of high-traffic volumes and highly restrictive design environments. Ramps are typically provided with geometries at or very near minimum design values because of the high cost of right-of-way (ROW) and the high cost of providing bridge structures. Providing a ramp designed at minimum values is not problematic in and of itself because of the large factors of safety generally present in even a "minimum" level design. Challenges may arise, however, if safety or operational improvements are needed because the same restrictions present in the original design (plus, perhaps, restrictions present as a result of new structures built or proposed for construction subsequently) prevent easy modifications.

To provide the researchers with additional information regarding "typical" sight distance impediments in the urban environment, three field investigations were undertaken. These investigations focused on on-ramps with sight distance limitations.

CASE STUDY A

The first case study was a direct-connect ramp at an urban interchange of two access-controlled freeways. A photograph of the ramp, designed in 1987, is shown in Figure 3-1. The design, constrained by the complex geometry inherent in a multilevel urban interchange, is bordered by a retaining wall on the inside of the curve. Other aspects of the ramp are typical of a high-standard interchange: concrete barriers, metal beam guard fence, and safety end treatments are provided at obstacles and drop-offs; drainage details and cut- and fill-slopes appear to be acceptable; and vertical curvature rates are relatively modest.

The horizontal curve radius and offset distance were reviewed to evaluate available sight distance on the ramp. The applicable sight distance criteria is stopping sight distance (SSD), which should be provided at every point along an alignment. The value of SSD used, however, is contingent on the design speed selected for use by the designer. Three different values for design speed are suggested in the 1994 *Green Book*[2] and the *Design Manual*,[1] representing various percentages of the design speed on the connecting highway. These values are characterized as upper range (85 percent), middle range (70 percent), and lower range (50 percent), and are shown in Table 3-1. Although the exact design speeds actually used by the designers on the connecting highways were not available, desirable (110 km/h) and minimum (80 km/h) design speeds used by TxDOT for controlled access facilities were used for the purposes of this analysis.

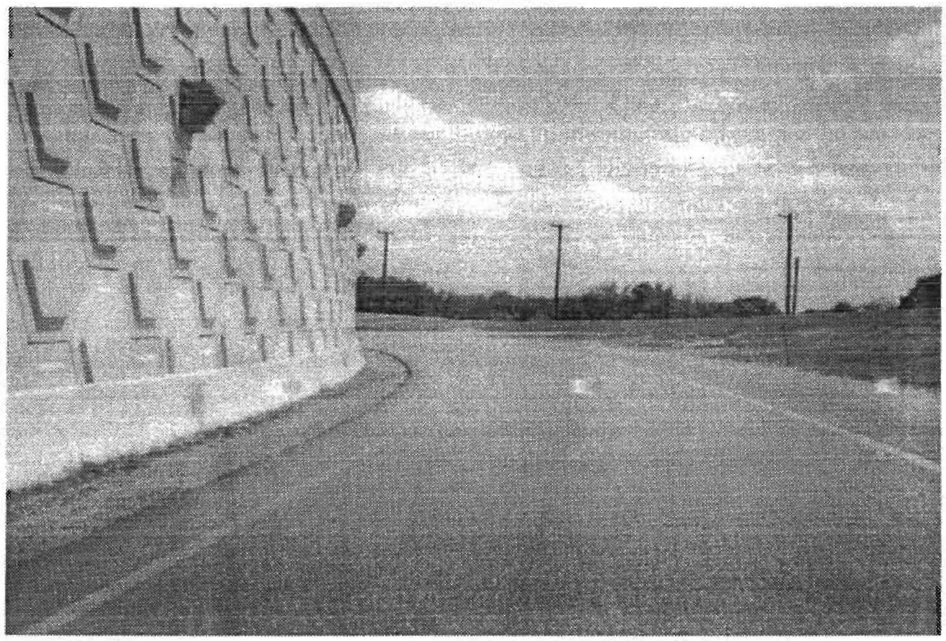


Figure 3-1. Case Study A: Direct Connect Ramp

Table 3-1. Summary, Case Study A

Design Speed Criteria	Assumed Design Speed (km/h)	Calculated Operating Speed Percentile	Minimum Calculated Offset (m)	Desirable Calculated Offset (m)	Available Offset (As-Built) (m)
Based on 110 km/h Main Lane Design Speed:					
Upper Range (85%)	100	>99	16.28	27.48	7.62
Middle Range (70%)	80	85	8.47	12.88	7.62
Lower Range (50%)	60	3	3.69	4.78	7.62
Based on 80 km/h Main Lane Design Speed:					
Upper Range (85%)	70	43	8.17	8.17	7.62
Middle Range (70%)	60	3	3.69	4.78	7.62
Lower Range (50%)	40	<1	1.32	1.32	7.62
85th Percentile Operating Speed = 80 km/h			8.47	12.88	7.62

To provide a frame of reference for the ramp, a spot speed study was conducted to determine the operating speed on the ramp. Speeds of free-flow vehicles were measured in approximately the middle of the ramp. The plot of the data is shown in Figure 3-2. The 85th percentile operating speed (listed in Table 3-1) was 80 km/h, and is provided as a comparison to the different design speeds that could have been used to design the ramp. In addition, Table 3-1 lists the operating speed percentiles that the assumed design speeds represent. Using Equation 9 for offsets to visual obstructions on the inside of horizontal curves, both minimum and desirable offset distances were calculated based on minimum and desirable SSD for the various design speed values.

When comparing operating speed to design speed, a ramp design speed of 60 km/h corresponds to approximately the third percentile value of speeds measured on the ramp. Although design speed and the appropriateness of the design criteria shown in the 1994 *Green Book*[2] and the *Design Manual*[1] are outside the bounds of this study, it appears that the use of the lower range design speeds would be questionable in this case. If this lower range design speed were used for design purposes, however, the available sight distance on the ramp provided by the offset (7.62 m) would exceed the offsets calculated using both minimum and desirable SSD requirements (3.69 and 4.78 m, respectively). As shown in Table 3-1, the as-built offset exceeded the values associated with design speeds of 40 to 60 km/h.

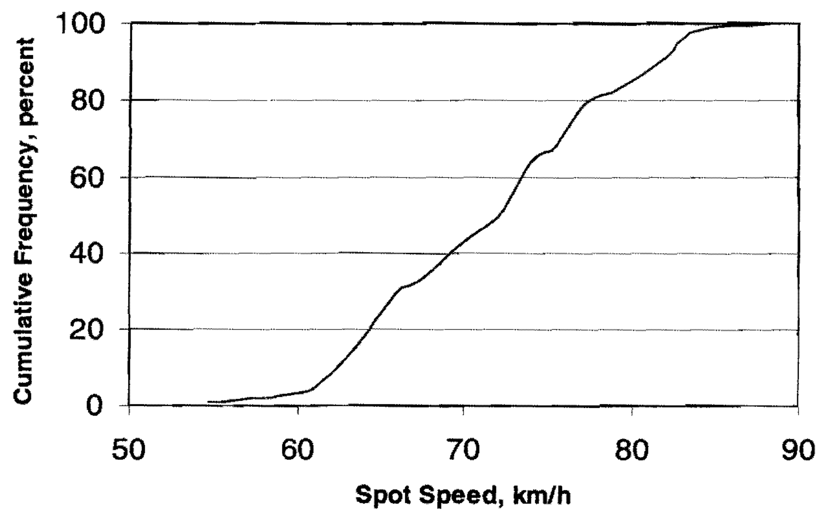


Figure 3-2. Spot Speed Study, Case Study A

The middle range design speed based on a design speed of 110 km/h on the main lanes corresponded to approximately the 85th percentile speed. This speed appears to represent a more realistic design condition in this case. For example, an offset calculated using desirable SSD requirements would exceed that value available by 5.26 m; however, calculated using minimum SSD requirements, the offset would exceed that width available by only 0.85 m.

Ramp and direct connection shoulder width design values used by TxDOT[1] are shown in Table 3-2. These widths, based on widths presented in the 1994 *Green Book*,[2] provide a total shoulder width of 2.4 to 3.6 m, allowing motorists to bypass stalled vehicles on one-lane ramps. An alternative design solution to this existing ramp could be to provide a narrower outside shoulder and correspondingly wider inside shoulder, providing the additional 0.85 m offset. This would provide greater sight distance and still permit bypassing stalled vehicles on the ramp.

Table 3-2. Ramp and Direct Connection Shoulder Widths[1]

Design Criteria	Inside Shoulder Width	Outside Shoulder Width
Minimum	0.6 m (Uncurbed Roadway) 1.2 m (Structures and Curbed Roadways)	1.8 m
Desirable	—	2.4 m

CASE STUDY B

The second case study examined the on-ramp shown in Figures 3-3 and 3-4. The ramp connects the local street network to a limited access freeway. The ramp design was complicated by two factors: an historic structure limiting the availability of right-of-way and a harbor bridge that provides clearance for large ocean-going ships. Both the horizontal and vertical curvature limited the available sight distance on the ramp. The small offset distance available on the inside of the horizontal curve on the ramp was caused by the concrete parapet at the top of a retaining wall; the retaining wall was necessary to prevent encroachment on the historic structure. The sharp vertical curvature was dictated by the large grade difference and short horizontal distance between the surface street network and the approach to the harbor bridge.

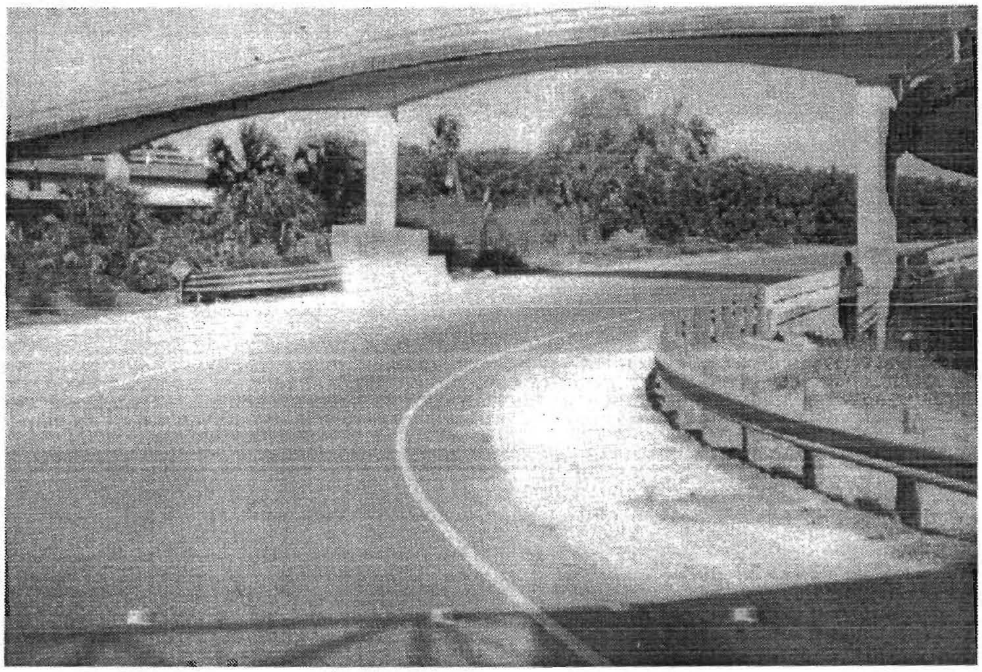


Figure 3-3. Case Study B, Horizontal Curvature

Figure 3-4. Case Study B, Vertical Curvature

Horizontal Curvature

Similarly to case study A, alternative design speeds were examined to compare resulting offset distances to those actually provided for the on-ramp. These design speeds (based again on assumed 110 km/h and 80 km/h main lane design speeds) are shown in Table 3-3. They ranged from 40 to 100 km/h; the 85th percentile speed (distribution shown in Figure 3-5) was approximately 50 km/h. The existing offset for the horizontal curve at the study site (5.82 m) exceeded the value calculated (5.12 m) using the lower range design speed of 40 km/h (based on a main lane design speed of 80 km/h). This offset corresponded to the 32nd percentile operating speed. Meeting the minimum calculated offset for the 85th percentile operating speed would require an additional 2.63 m, while meeting the desirable calculated offset would require an additional 4.24 m.

Table 3-3. Summary, Horizontal Curve for Case Study B

Design Speed Criteria	Assumed Design Speed (km/h)	Calculated Operating Speed Percentile	Minimum Calculated Offset (m)	Desirable Calculated Offset (m)	Available Offset (As-Built) (m)
Based on 110 km/h Main Lane Design Speed:					
Upper Range (85%)	100	>99	51.73	73.88	5.82
Middle Range (70%)	80	>99	29.86	42.75	5.82
Lower Range (50%)	60	99	13.87	17.71	5.82
Based on 80 km/h Main Lane Design Speed:					
Upper Range (85%)	70	>99	21.56	28.98	5.82
Middle Range (70%)	60	99	13.87	17.71	5.82
Lower Range (50%)	40	32	5.12	5.12	5.82
85th Percentile Operating Speed = 50 km/h			8.45	10.06	5.82

Widening the inside shoulder would partially alleviate the sight distance constraints imposed by the concrete parapet on the retaining wall, allowing the site to meet the 85th percentile operating speed minimum offsets. Because the site has a larger than required shoulder on both the inside and outside of the curve, this width increase could be accomplished without moving the retaining wall. Meeting either the middle range or upper range criteria would require moving the retaining wall or changing the typical section to eliminate the concrete barrier. Either modification would be extremely costly and politically difficult to accomplish.

Vertical Curvature

Sight distance at the case study B site was also constrained by the vertical curvature used in the design. As shown in Table 3-4, the crest vertical curve used at the apex of the ramp did not meet SSD requirements for the design speeds reviewed. The curve length provided was 7 m shorter than required for the lowest design speed of 40 km/h.

Solutions to the concerns with the site's vertical alignment are difficult to derive. A longer vertical curve would overlap with the sag curve prior to the crest curve and prevent attaining a tie-in with the grades on the freeway. Raising the grade on the local street network would require rebuilding a number of at-grade intersections and streets and reduce the clearance at one of the freeway structures, while lowering the grade on the freeway would reduce clearance for the harbor bridge and also reduce clearance over the local street network.

Table 3-4. Summary, Vertical Curve for Case Study B

Design Speed Criteria	Design Speed (km/h)	Calculated Operating Speed Percentile	Minimum Curve Length (m)	Desirable Curve Length (m)	Actual Curve Length (m)
Based on 110 km/h Main Lane Design Speed:					
Upper Range (85%)	100	>99	703	1192	49
Middle Range (70%)	80	>99	361	551	49
Lower Range (50%)	60	99	157	203	49
Based on 80 km/h Main Lane Design Speed:					
Upper Range (85%)	70	>99	251	348	49
Middle Range (70%)	60	99	157	203	49
Lower Range (50%)	40	32	56	56	49
85th Percentile Operating Speed = 50 km/h			94	112	49

CASE STUDY C

Case study C reviewed the design of an on-ramp connecting a frontage road to an exit ramp from a freeway. The relatively unusual ramp location resulted from the stage construction of the multiple freeways in the immediate vicinity, with the ramp providing needed access that has since become redundant. The ramp, now permanently closed, provided problematic operation and an unacceptably high accident rate while it was in operation.

Reviewing sight distance restrictions at the site, attention focused on the vertical alignment. A crest and a sag vertical curve are present on the ramp, which descends from the frontage road to merge with the exit ramp (see Figures 3-6 and 3-7). As summarized in Tables 3-5 and 3-6, an examination revealed that the design for each curve met lower range design speed desirable length criteria for both 80 and 110 km/h design speeds. Comparisons regarding the 85^{th} percentile operating speed were not available because of the ramp's closure.

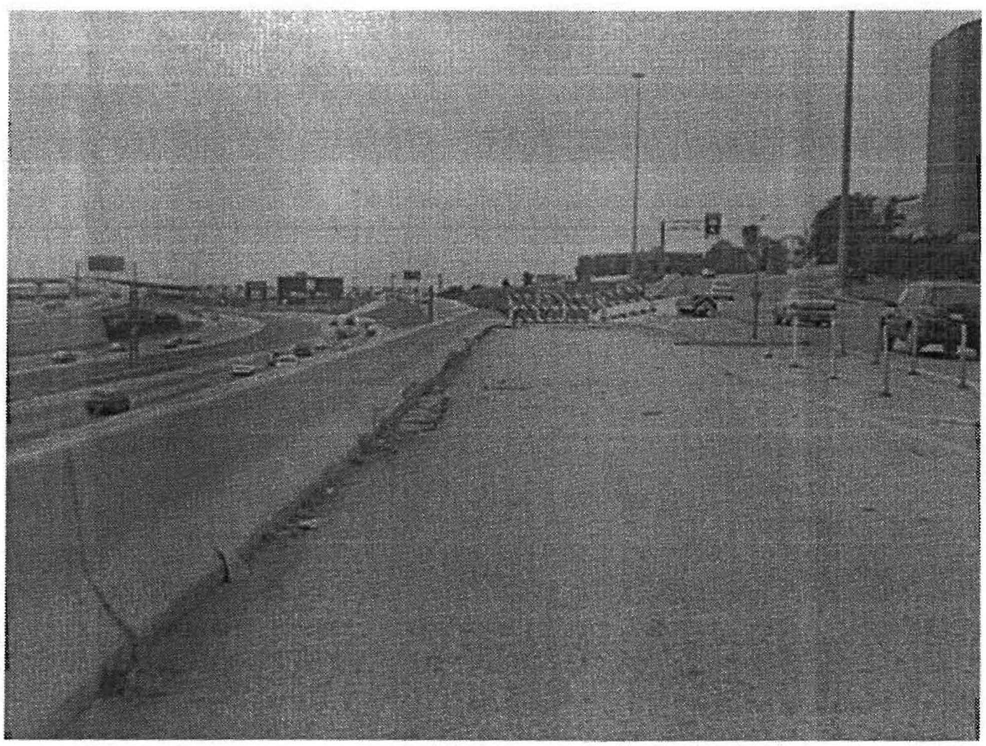


Figure 3-6. Crest Vertical Curve, Case Study C

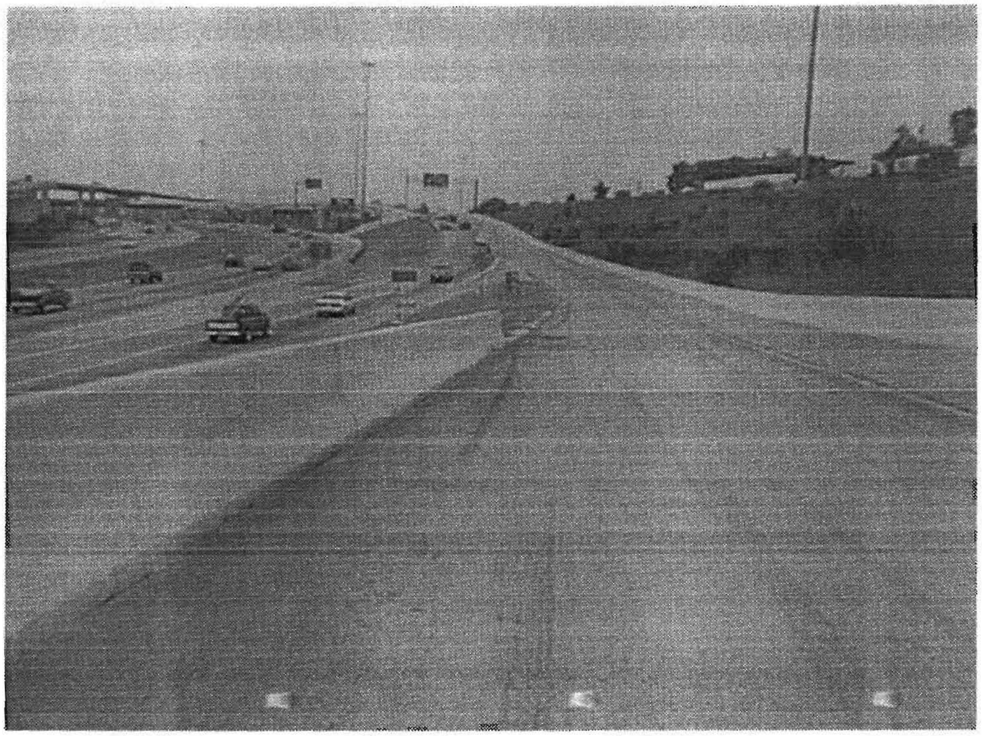


Figure 3-7. Sag Vertical Curve, Case Study C

Sight distance on the vertical curves present at the site appeared to be relatively good, although it would have been desirable to compare operating speeds on the ramp with the design speeds presented. Reasons for the poor performance of the ramp could be charged to several issues (and have indeed been the subject of an extensive investigation). Modifications to the vertical alignment of the ramp to improve available sight distance would be possible, although relatively expensive due to the rolling terrain. The improvement might have improved the performance of the facility, although the efficacy of such measures appears doubtful given the relatively good design standard already in place.

Table 3-5. Summary, Crest Vertical Curve for Case Study C

Design Speed Criteria	Design Speed (km/h)	Minimum Curve Length (m)	Desirable Curve Length (m)	Actual Curve Length (m)
Based on 110 km/h Main Lane Design Speed:				
Upper Range (85%)	100	249	345	122
Middle Range (70%)	80	197	214	122
Lower Range (50%)	60	85	111	122
Based on 80 km/h Main Lane Design Speed:				
Upper Range (85%)	70	155	223	122
Middle Range (70%)	60	99	125	122
Lower Range (50%)	40	39	39	122

Table 3-6. Summary, Sag Vertical Curve for Case Study C

Design Speed Criteria	Design Speed (km/h)	Minimum Curve Length (m)	Desirable Curve Length (m)	Actual Curve Length (m)
Based on 110 km/h Main Lane Design Speed:				
Upper Range (85%)	100	221	293	113
Middle Range (70%)	80	154	194	113
Lower Range (50%)	60	77	111	113
Based on 80 km/h Main Lane Design Speed:				
Upper Range (85%)	70	153	164	113
Middle Range (70%)	60	115	134	113
Lower Range (50%)	40	61	61	113

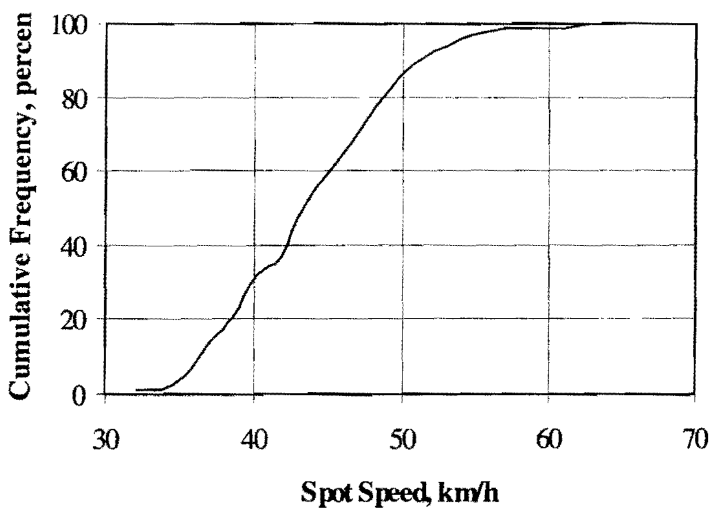


Figure 3-5. Spot Speed Study, Case Study B.

CONCLUSIONS

The case studies examined revealed that appropriate design criteria are somewhat difficult to discern in advance of the construction of a facility. Care must be taken to select appropriate design speeds that yield acceptable designs. Design speeds that, although acceptable according to guidelines, only accommodate a small percentage of drivers are problematic and inappropriate. Of course, the difficulty lies in accurately predicting which design speeds *are* appropriate.

Critiquing a design from the viewpoint of information not available to the designer at the time the design was completed (i.e., using a spot speed study to calculate an 85th percentile speed) is somewhat unrealistic and limited in application. It does, however, lead to the desire to have additional guidance about the selection of ramp design speed in general. Reasonably accurate predictions about operating speeds and design conditions on ramps would be helpful to the designer as ramp curvature and sight distance requirements are selected for use in the design of an interchange. However, ramp design speed lies outside the bounds of this study and is currently the subject of other TxDOT sponsored research.

Clearly, the central issue remains that designers and planners must recognize at an early point in the design of a facility that sight distance is of great importance in the operation of that facility. Both vertical and horizontal curvature must be examined with regard to their impacts on sight distance. Although some modifications can be made to improve available sight distance at some locations, frequently those modifications are either extremely expensive or virtually impossible to effect once the facility is in place.

CHAPTER 4

RECOMMENDATIONS FOR IMPLEMENTATION

This chapter provides proposed revisions to the *Design Manual.*[1] It is recommended that these revisions be incorporated into the current rewrite of the *Design Manual* so that they become effective when the *Design Manual* is republished. Existing TxDOT training courses may also be used to inform designers of the proposed changes to the *Design Manual.*

The *Design Manual* communicates recommended design practices, procedures, and criteria to highway designers and engineers. Although other references and design materials are frequently and necessarily used by designers and engineers at TxDOT, this manual provides guidance regarding geometric design.

The *Design Manual* has a number of references to sight distance in various sections, but substantive guidance is provided only with regard to stopping sight distance and passing sight distance. Material related to intersection sight distance is scant, and decision sight distance is not mentioned in the manual.

Stopping sight distance criteria are provided in the section on Basic Design Criteria. In this section, a table presents minimum and desirable stopping sight distance values for design speeds from 30 to 120 km/h. The minimum and desirable values reflect the range of stopping sight distance values given in the 1994 *Green Book.*[2] The values in the *Design Manual*, however, are rounded up to the nearest value of 10. In some instances, this results in a recommended value 9 m higher than what is recommended in the *Green Book.*

Stopping sight distance is also addressed in the section on freeways in the discussion of sight distance on ramps. In this section, the manual maintains that stopping sight distance is provided along ramps from the terminal junctions along the freeway.

Intersection sight distance is mentioned very briefly in the discussions on intersections in each of the following sections of the manual: Urban Streets, Multilane Rural Highways, and Two-Lane Rural Highways. In these sections, general statements are provided regarding intersection sight distance and directing the reader to relevant sections of the manual.

Specific recommended changes to the text of the *Design Manual* follow. A brief discussion of the reason for the changes is included with suggested wording for the recommended modifications. Figure and table numbers within the material quoted from the *Design Manual* refer to the figures and tables as numbered within the *Design Manual.* Figures and tables are provided only where proposed for modification (i.e., figures or tables extraneous to any proposed modifications are not included).

BASIC DESIGN CRITERIA, DESIGN ELEMENTS, Sight Distance

In this section of the manual, sight distance is introduced with a brief rationale for its importance. The only two types of sight distance previously mentioned, however, are stopping sight distance and passing sight distance. Therefore, decision sight distance and intersection sight distance are added to the discussion.

> Of utmost importance in highway design is the arrangement of geometric elements so that there is adequate sight distance for safe and efficient traffic operation assuming adequate light, clear atmospheric conditions, and drivers' visual acuity. For design, ~~two types of sight distance are considered: that for overtaking vehicles, and that required for stopping. Passing sight distance is applicable only in the design of two-lane rural highways and therefore is presented in Paragraph 4-502(D)~~ four types of sight distance are considered:
>
> - Stopping sight distance;
> - Decision sight distance;
> - Passing sight distance; and
> - Intersection sight distance.

BASIC DESIGN CRITERIA, DESIGN ELEMENTS, Sight Distance, Stopping Sight Distance

This section defines stopping sight distance and includes the criteria for its application. Table 4-1 [*Design Manual* Figure 4-3] provides distances to be used for various design speeds. Previously, a very conservative hard conversion from English to metric units was used, providing up to an 11 percent increase in required sight distance. Because the values provided meet current design criteria and are already quite conservative, it is recommended that values are rounded to the nearest meter rather than to the nearest ten meters.

Minimum stopping sight distance is the length of roadway required to enable a vehicle traveling at or near design speed to safely stop before reaching an object in its path. Minimum stopping sight distance values for various design speeds are tabulated in Figure 4-3 [research report Table 4-1]. These values are based on driver's eye height of 1,070 mm and object height of 150 mm as recommended by AASHTO. After selection of design speed, stopping sight distance values become a controlling element for several basic design features such as roadway alignment and non-signalized intersection design. Greater than minimum stopping sight distances should normally be used, and minimum values should be used only in select instances where economic or other restrictive conditions dictate (see Minimum Standards).

Table 4-1. Recommended Changes to *Design Manual* Figure 4-3

Figure 4-3. Stopping Sight Distance Values (Wet Pavements).		
Design Speed (km/h)	Stopping Sight Distance (m)	
	Minimum	Desirable
30	30	30
40	~~50~~ 45	~~50~~ 45
50	~~60~~ 58	~~70~~ 63
60	~~80~~ 75	~~90~~ 85
70	~~100~~ 95	~~120~~ 111
80	~~120~~ 113	140
90	~~140~~ 132	~~170~~ 169
100	~~160~~ 157	~~210~~ 205
110	180	~~250~~ 247
120	~~210~~ 203	~~290~~ 286

BASIC DESIGN CRITERIA, DESIGN ELEMENTS, Sight Distance, Decision Sight Distance

It is recommended that a section defining decision sight distance be provided to designers. Criteria for its use should also be included.

Decision sight distance is the distance required for a driver to detect an unexpected or otherwise difficult-to-perceive information source, recognize the source, select an appropriate speed and path, and initiate and complete the required maneuver safely and efficiently. Because decision sight distance gives drivers additional margin for error and affords them sufficient length to maneuver their vehicles at the same or reduced speed rather than to just stop, its values are substantially greater than stopping sight distance. Table 4-2 [research report Figure 4-2] shows recommended decision sight distance values for various avoidance maneuvers.

Examples of situations in which decision sight distance is preferred include the following:

- Interchange and intersection locations where unusual or unexpected maneuvers are required (such as exit ramp gore areas and left-hand exits);
- Changes in cross section such as toll plazas and lane drops; and
- Areas of concentrated demand where there is apt to be "visual noise" whenever sources of information compete, as those from roadway elements, traffic, traffic control devices, and advertising signs.

Table 4-2. Recommended New Table

Table. Recommended Decision Sight Distance Values					
Design Speed (km/h)	Decision Sight Distance (m)				
	Avoidance Maneuver [1]				
	A	B	C	D	E
50	75	160	145	160	200
60	95	205	175	205	235
70	125	250	200	240	275
80	155	300	230	275	315
90	185	360	275	320	360
100	225	415	315	365	405
110	265	455	335	390	435
120	305	505	375	415	470

[1] A: Stop on rural road.
B: Stop 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.

BASIC DESIGN CRITERIA, DESIGN ELEMENTS, Sight Distance, Intersection Sight Distance

A section defining intersection sight distance and the rationale for its use is recommended for inclusion in the manual. Full development of all of the AASHTO[2] models within the TxDOT manual was judged to be unnecessary because no changes from AASHTO guidelines were envisioned as necessary. Accordingly, designers are made aware of intersection sight distance and referred to AASHTO's *A Policy on Geometric Design of Highways and Streets.*

The operator of a vehicle approaching an intersection at grade should have an unobstructed view of the entire intersection and an adequate view of the intersecting highway to permit control of the vehicle to avoid a collision. When designing an intersection, the following factors should be taken into consideration:

- Adequate sight distance should be provided along both highway approaches and across corners.
- Gradients of intersecting highways should be as flat as practical on sections that are to be used for storage of stopped vehicles.
- Combination of vertical and horizontal curvature should allow adequate sight distance of the intersection.
- Traffic lanes should be clearly visible at all times.
- Lane markings and signs should be clearly visible and understandable from a desired distance.
- Intersections should be free from the sudden appearance of potential conflicts.

For selecting appropriate intersection sight distance, refer to ***A Policy on Geometric Design for Streets and Highways***, AASHTO. Sight distance criteria are provided for the following five types of intersection controls:

1. No control, but allowing vehicles to adjust speed;
2. Yield control on minor roads;
3. Stop control on minor roads;
4. Signal control; and
5. Stopped vehicle turning left from major highway.

BASIC DESIGN CRITERIA, DESIGN ELEMENTS, Sight Distance, Sight Distance on Horizontal Curves

In this section, material has been added to include an equation developed by P. L. Olson, et al. This equation conservatively approximates the required offset for sight obstructions on horizontal curves. Additional material has been provided directing readers to AASHTO's section on measuring sight distance graphically. Although numerous equations have been developed to calculate sight distance for a wide variety of conditions, graphical methods can in most cases more readily accommodate the needs of the designer where sight obstructions are near the ends of horizontal curves or where unusual combinations of curves are encountered. Finally, the figure providing offset requirements for cases where sight distance is less than the length of horizontal curve was updated to conform with the text of the manual.

Where an object off the pavement, such as bridge pier, bridge railing, median barrier, building, cut slope, or natural growth restricts sight distance, the minimum radius of curvature is determined by the stopping sight distance.

Stopping sight distance on horizontal curves is obtained from the following equations:

$$S<L: \qquad M = R[1 - Cos\frac{28.65S}{R}]$$

where: L = Curve length

M = Middle ordinate

S = Stopping sight distance

R = Radius

For S>L (from P. L. Olson et al., NCHRP Report 270, Parameters Affecting Stopping Sight Distance, Transportation Research Board, Washington, DC, June 1984):

$$S>L: \qquad M=\frac{L(2S-L)}{8R}$$

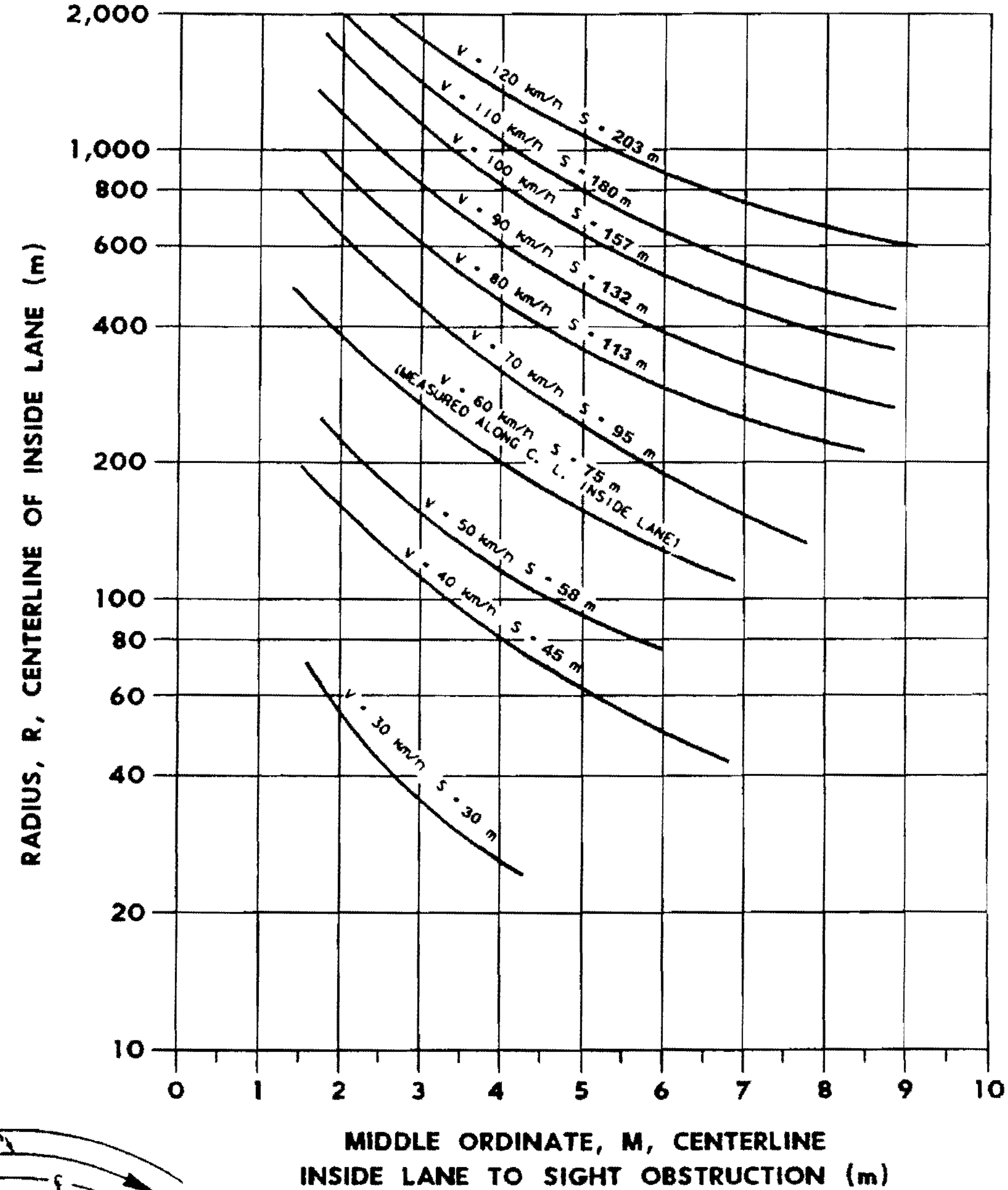
where: L	=	Curve length
M	=	Middle ordinate
S	=	Stopping sight distance
R	=	Radius

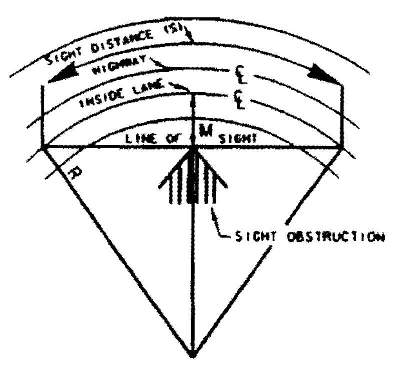
Figure 4-12 [research report Figure 4-1] provides a graph illustrating the required offset where stopping sight distance is less than the length of curve (S<L).

Figure 4-12 [research report Figure 4-1] may be used in either case but may be overly conservative for curves with small deflection angles. In cases where complex geometries or discontinuous objects cause sight obstructions, graphical methods may be useful in determining available sight distance and associated offset requirements.

It is assumed that the driver's eye is 1,070 mm above the center of the inside lane (inside with respect to curve) and the object is 150 mm high. The line-of-sight is assumed to intercept the view obstruction at the midpoint of the sight line and 600 mm above the center of the inside lane. The clear distance is measured from the center of the inside lane to the obstruction.

To check horizontal sight distance on the inside of a curve graphically, sight lines equal to the required sight distance on horizontal curves should be reviewed to ensure that obstructions such as buildings, hedges, barrier railing, high ground, etc., do not restrict sight below that required in either direction. See **AASHTO**'s Chapter III section on measuring and recording sight distance on plans in Chapter 3 for further information.





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

WHERE
S = STOPPING SIGHT DISTANCE
M = MIDDLE ORDINATE
R = RADIUS

Figure 4-1. Revisions to *Design Manual* Figure 4-12. Stopping Sight Distance on Horizontal Curves

(Updated to conform with SSD values used in text)

NEW LOCATION AND RECONSTRUCTION (4R) DESIGN CRITERIA, URBAN STREETS, Intersections

A brief mention of intersection sight distance is provided to direct the designer to the appropriate section of the manual.

> The number, design, and spacing of intersections influence the capacity, speed, and safety on urban streets. Capacity analysis of signalized intersections is one of the most important considerations in intersection design. Dimensional layout or geometric design considerations are closely influenced by traffic volumes and operational characteristics and the type of traffic control measures used.
>
> Because of the space limitations and lower operating speeds on urban streets, curve radii for turning movements are less than for rural highway intersections. Curb radii of 4.5 m to 7.5 m permit passenger cars to negotiate right turns with little or no encroachment on other lanes. Where heavy volumes of trucks or buses are present, increased curb radii of 9 m to 15 m expedite turns to and from through lanes. Where combination tractor-trailer units are anticipated in significant volume, reference should be made to the material in section, MINIMUM DESIGNS FOR TRUCK AND BUS TURNS.

In general, intersection design should be rather simple, and free of complicated channelization, to minimize driver confusion. Sight distance is an important consideration even in the design of signalized intersections since, during the low volume hours, flashing operation may be used. <u>For more information on sight distance as part of intersection design, see Intersection Sight Distance Design, Chapter II.</u>

NEW LOCATION AND RECONSTRUCTION (4R) DESIGN CRITERIA, TWO-LANE RURAL HIGHWAYS, Intersections

Text is added to this section recommending general design practices to be followed in the vicinity of intersections, together with references to material on intersection sight distance.

The provision of adequate sight distance is of utmost importance in the design of intersections along two-lane rural highways. At intersections, consideration should be given to avoiding steep profile grades and locating intersections on or near a short crest vertical curve or a sharp horizontal curve. These locations could result in poor operations and/or inadequate sight distance at the intersection. Where necessary, backslopes should be flattened and horizontal and vertical curves lengthened to provide additional sight distance. For more information on intersection sight distance, see **Intersection Sight Distance**, Chapter II.

Desirably, the roadways should cross at approximately right angles. Where crossroad skew is flatter than 60 degrees to the highway, the crossroad should be re-aligned to provide for a near perpendicular crossing. The higher the functional classification, the closer to right angle the crossroad intersection should be.

Section 4-710 provides information regarding the accommodation of various types of truck class vehicles in intersection design. Further information on intersection design may also be found in AASHTO's **A Policy on Geometric Design of Highways and Streets.**

NEW LOCATION AND RECONSTRUCTION (4R) DESIGN CRITERIA, MULTILANE RURAL HIGHWAYS, INTERSECTIONS

Reference is made to sight distance sections in appropriate locations. Further recommendations are made for modifications at median openings to improve operations at those points.

In the design of intersections, careful consideration should be given to the appearance of the intersection from the driver's perspective. In this regard, design should be rather simple to avoid driver confusion. In addition, adequate sight distance should be provided throughout, especially in maneuver or conflict areas. See section on **Sight Distance** in Chapter II for further information regarding sight distance.

Right-angle crossings are preferred to skewed crossings, and where skew angles exceed 60 degrees, alignment modifications are generally necessary. Speed change lanes may be provided in accordance with the previous discussion in the section on **Speed Change Lanes**.

Section 4-710 provides information regarding the accommodation of various types of truck class vehicles in intersection design. AASHTO's **A Policy on Geometric Design of Highways and Streets** should be consulted for further information on intersection design and intersection sight distance.

Intersections formed at bypass and existing route junctions should be designed so as not to mislead drivers as typified in **Figure 4-35**.

For intersections with a narrow, depressed median section, it may be necessary to have superelevation across the entire cross section to provide for safer operation at the median openings.

For more information on intersection design, see **Intersection Design**, Chapter II.

NEW LOCATION AND RECONSTRUCTION (4R) DESIGN CRITERIA, FREEWAYS

Modified text is provided to allow reversing the inside and outside shoulder width recommendations for bridges if sight distance requirements may be met in this manner. Providing a larger inside shoulder will in some cases provide improved sight distance without moving the structures. The combination of the inside and outside shoulder widths would still permit passing a stalled vehicle. Table 4-3 contains the recommended revisions to the *Design Manual*'s Figure 4-51.

Table 4-3. Revisions to Figure 4-51 in the *Design Manual*

<table>
<tr><th colspan="8">Figure 4-51. Roadway and Structure Widths for Controlled Access Facilities.</th></tr>
<tr><th>Type of Roadway</th><th colspan="2">Inside Shoulder Width (m)</th><th colspan="2">Outside Shoulder Width[1] (m)</th><th>Traffic Lanes (m)</th><th colspan="2">Structure Width (m)</th></tr>
<tr><td>Mainlanes:</td><td colspan="2"></td><td colspan="2"></td><td></td><td colspan="2"></td></tr>
<tr><td>4-Lane Divided</td><td colspan="2">1.2</td><td colspan="2">3.0</td><td>7.2</td><td colspan="2">11.4 Min.</td></tr>
<tr><td>6-Lane Divided</td><td colspan="2">3.0[1]</td><td colspan="2">3.0</td><td>10.8</td><td colspan="2">16.8 Min.[2]</td></tr>
<tr><td>8 Lanes or More</td><td colspan="2">3.0</td><td colspan="2">3.0</td><td>14.4[3]</td><td colspan="2">20.4 Min.[3]</td></tr>
<tr><td>1-Lane Direct Conn.[7]</td><td>0.6 Rdwy.
1.2 Str.</td><td>See
Note 4</td><td colspan="2">2.4</td><td>4.2</td><td colspan="2">7.8 Min.</td></tr>
<tr><td>2-Lane Direct Conn.</td><td>0.6 Rdwy.
1.2 Str.</td><td>See
Note 4</td><td colspan="2">2.4</td><td>7.2</td><td colspan="2">10.8 Min.</td></tr>
<tr><td rowspan="2">Ramps[7]</td><td>0.6 Rdwy.</td><td>See</td><td>Min.</td><td>Des.</td><td rowspan="2">4.2</td><td>Min.</td><td>Des.</td></tr>
<tr><td>1.2 Str.</td><td>Note 4</td><td>1.8</td><td>2.4[5]</td><td>7.2</td><td>7.8[5]</td></tr>
<tr><td rowspan="2">Median Width</td><td colspan="2">Urban</td><td>7.2[6]
(usual)</td><td>--</td><td rowspan="2"></td><td>--</td><td>--</td></tr>
<tr><td colspan="2">Rural</td><td>14.4</td><td>22.5</td><td>--</td><td>--</td></tr>
</table>

[1] Minimum 3.0 m inside shoulders are usually provided on urban freeways with flush medians and six or more lanes. For urban freeway rehabilitation and expansion, the provision of wide inside shoulders may not be feasible. Under these circumstances documentation for narrower shoulders should be submitted and a design exception requested. Six-lane freeways with depressed median may include 1.2 m shoulders with 15.0 m minimum structure width.

[2] For auxiliary (speed change) lanes, see Main Lanes, **Shoulders** for outside shoulder width.

[3] For more than eight lanes, add 3.6 m width per lane.

[4] Minimum inside shoulder width is 0.6 m on uncurbed roadway sections and 1.2 m on bridges and curbed roadways. All longitudinal traffic barriers, including bridge rail, wall, and guard fence, should be located a minimum of 1.2 m from the travel lane edge.

[5] Desirable values should be used where there are sufficient combination type vehicles to govern design. Where ramp ADT includes greater than 10 percent trucks, desirable values are appropriate for use.

[6] Applicable to urban freeways with flush medians and six or more mainlanes.

[7] <u>If sight distance restrictions are present due to horizontal curvature, the shoulder width on the inside of the curve may be increased and the shoulder width on the outside of the curve decreased to 0.6 m (Rdwy) or 1.2 m (Str).</u>

NEW LOCATION AND RECONSTRUCTION (4R) DESIGN CRITERIA, FREEWAYS, Sight Distance

Text is provided to indicate the need for increased sight distance on a freeway prior to an exit ramp. Decision sight distance or, alternatively, a 25 percent increase in stopping sight distance, is recommended to the designer for this situation.

> On all ramps and direct connections, the combination of grade, vertical curves, alignments, and clearance of lateral and corner obstructions to vision shall be such as to provide sight distance along such ramps and connections from terminal junctions along the freeway, consistent with the probable speeds of vehicle operation. **Figure 4-55** shows recommended minimum and desirable stopping sight distances for ramps and direct connections.
>
> <u>The sight distance on a freeway preceding the approach nose of an exit ramp should exceed the minimum stopping sight distance for the freeway design speed, preferably by 25 percent or more. Decision sight distance, as discussed in Chapter II, **Decision Sight Distance**, is a desirable goal.</u>

LONGITUDINAL BARRIERS, CONCRETE BARRIERS, LOCATION

Text is provided to indicate to the designer that the provision of concrete barriers may impede sight distance on horizontal curves.

> On controlled access highways, concrete barriers will generally be provided in medians of 9.0 m or less. On non-controlled access highways, concrete barriers may be used on medians of 9.0 m or less; however, care should be exercised in their use in order to avoid the creation of an obstacle or restriction in sight distance at median openings or on horizontal curves. Generally, the use of concrete barriers on non-controlled access facilities should be restricted to areas with potential safety concerns such as railroad separations or through areas where median constriction occurs.

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