NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP Report 383

Intersection Sight Distance

Transportation Research Board
National Research Council

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Subject Areas

Highway and Facility Design
Highway Operations, Capacity and Traffic Control
Safety and Human Performance

Research Sponsored by the American Association of State Highway and Transportation Officials in Cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY PRESS
Washington, D.C. 1996

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NCHRP REPORT 383

Project 15-14(1) FY '92

ISSN 0077-5614

ISBN 0-309-06051-6

L. C. Catalog Card No. 96-61607

Price $28.00

NOTICE

The project that is the subject of this report was a part of the National Cooperative Highway Research Program conducted by the Transportation Research Board with the approval of the Governing Board of the National Research Council. Such approval reflects the Governing Board's judgment that the program concerned is of national importance and appropriate with respect to both the purposes and resources of the National Research Council.

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Each report is reviewed and accepted for publication by the technical committee according to procedures established and monitored by the Transportation Research Board Executive Committee and the Governing Board of the National Research Council.

Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

Printed in the United States of America

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Printed in the United States of America

FOREWORD

By Staff
Transportation Research Board

This report describes the development of recommended revisions to the intersection sight distance design policy that appears on pages 696 through 721 of the 1994 AASHTO Green Book, *A Policy on Geometric Design of Highways and Streets*. The contents of this report are, therefore, of immediate interest to highway and facility designers; highway-operations, capacity, and traffic-control personnel; and others concerned with safety and motorists' reactions. The report's conclusions are based on field observations of driver behavior at a variety of selected at-grade intersections, including those with no control, with minor-road Yield control, and with minor-road Stop control.

The current AASHTO intersection sight distance procedures are intended to provide adequate sight distance at intersections to promote safe and efficient traffic operations. The basic intersection sight distance (ISD) models for no control, Stop control, and signal control were formulated in 1940. Over the intervening years, the model parameters were modified to accommodate changes in the vehicle-driver-roadway system. Recently, questions have arisen about the validity of these models, as well as the appropriateness of certain parameter values used to calculate ISD.

Of particular concern, were the Case IIIB and Case IIIC design criteria (i.e., ISDs for a vehicle on a Stop-controlled approach on a minor road to accelerate from a stopped position and turn left or right, respectively, into the major road), which many considered excessive; and the Case I design criteria (i.e., ISD for vehicles approaching intersections with no control, at which vehicles are not required to stop, but may be required to adjust speed), which were based on assumptions that may not be sufficiently conservative.

The following are among the revised ISD concepts used by Midwest Research Institute in NCHRP Project 15-14(1), *Intersection Sight Distance*, to introduce a more consistent conceptual basis for ISD models and to set revised values for ISD design based on those models: the use of a revised stopping sight distance (SSD) model as the basis for all cases in which a vehicle approaching an intersection may need to stop or slow; the use of actual speed profiles that approaching drivers were observed using in the field at intersections with various types of traffic control; the use of the gap-acceptance behavior of drivers observed in the field as the basis for the leg of the departure triangle along the major road; and the use of vehicle-stopping positions observed in the field as the basis for the leg of the departure triangle along the minor road. Two alternative SSD models were considered as a basis for ISD policy: the current SSD model presented in the 1994 AASHTO Green Book and the revised SSD model developed in NCHRP Project 3-42, *Determination of Stopping Sight Distances*.

The recommended revisions to the AASHTO Green Book developed during this research were presented to the AASHTO Task Force on Geometric Design at their Summer, 1996 meeting as the first step towards possible AASHTO adoption. Those recommendations were reordered from the Case I through Case V format of the 1994 Green Book because the recommended sight distances for Yield-controlled intersections, currently

Case II, depend, in part, on the recommended model for Stop-controlled intersections, currently Case III.

Recommended sight distance policies are presented for the following situations:

- Case A. Intersections with no control,
- Case B. Intersections with Stop control on the minor road,
  - Case B1. Left turn from the minor road,
  - Case B2. Right turn from the minor road,
  - Case B3. Crossing maneuver from the minor road,
- Case C. Intersections with Yield control on the minor road,
  - Case C1. Crossing maneuver from the minor road,
  - Case C2. Left or right turn from the minor road,
- Case D. Intersections with traffic signal control,
- Case E. Intersections with all-way Stop control, and
- Case F. Left turns from a major road.

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ACKNOWLEDGMENTS

The work reported herein was performed under NCHRP Project 15-14(1) by Midwest Research Institute (MRI) and Pennsylvania Transportation Institute (PTI). The work was performed in MRI's Center for Regional Development, directed by Ms. Sandra A.J. Lawrence.

Mr. Douglas W. Harwood, Principal Traffic Engineer at MRI, was the principal investigator of the research. Other MRI staff members who contributed to the research include Ms. Karin M. Bauer, Mr. Robert R. Blackburn, Dr. William D. Glauz, Mr. Brian Rosson, Ms. Erin J. McGrane, and Mr. Thomas P. Grelinger. The subcontract work at PTI was directed by Dr. John M. Mason. Other PTI staff members who contributed to the work included Mr. Robert E. Brydia, Dr. Martin T. Pietrucha, Mr. Robert S. Hostetter, and Dr. Gary L. Gittings. Mr. Harwood, Dr. Mason, Mr. Brydia, Dr. Pietrucha, and Dr. Gittings served as coauthors of this final report.

The research team received valuable assistance from the staffs of three state highway agencies and two local highway agencies who participated in the study. The participating agencies were the Illinois Department of Transportation; the Missouri Highway and Transportation Department; the Pennsylvania Department of Transportation; the City of Phoenix, Arizona; and the City of Kansas City, Missouri. We are also grateful to highway design engineers in 107 state and local highway agencies who responded to a questionnaire concerning their intersection sight distance design practices.

INTERSECTION SIGHT DISTANCE

SUMMARY

The objective of this research was to evaluate driver needs for sight distance on approaches to at-grade intersections and to recommend appropriate revisions to current geometric design policies for intersection sight distance. The research scope included a range of types of intersection traffic control including intersections with no control, intersections with Stop control on the minor road, intersections with Yield control on the minor road, intersections with traffic signal control, and intersections with all-way Stop control.

Field studies to observe driver behavior were conducted at a total of 25 intersections located in four states. The field study intersections included uncontrolled, Stop-controlled, and Yield-controlled intersections.

The field studies at Stop-controlled intersections focused on the behavior of minor-road drivers waiting on the minor road for a gap in the major-road traffic stream in which to enter or cross the major road. It was found that drivers turning left or right from Stop-controlled approaches typically accept gaps in major-road traffic that are much shorter than the sight distances currently specified in the AASHTO Green Book. The observed gap-acceptance behavior in some cases requires deceleration by major-road vehicles at modest, comfortable rates to speeds that are typically about 70 percent of their upstream running speed.

Drivers on approaches to uncontrolled intersections on urban and suburban residential streets typically slow to about 50 percent of their midblock running speed, even if no potentially conflicting vehicles are present on an intersection approach. Similar driver behavior was observed on Yield-controlled approaches on urban and suburban residential streets, except that drivers typically slowed to only 60 percent of the midblock running speeds.

Field studies also showed that, where necessary to obtain a sufficient view of major-road traffic, vehicles on a Stop-controlled approach will move their vehicles closer to the road than the 3-m (10-ft) distance currently used in AASHTO policy. It was also found that, for most passenger vehicles, pick-up trucks, and minivans, the distance from the front of the vehicle to the driver's eye is 2.4 m (8.0 ft) or less.

The report recommends a new approach to intersection sight distance design policy that should provide a more consistent conceptual basis for intersection sight distance models. The clear sight triangles used at uncontrolled and Yield-controlled intersections should provide approaching vehicles sufficient sight distance to stop, if necessary, before reaching the intersection. It is recommended that the dimensions of such approach sight triangles should

be based on the same model used for stopping sight distance design, with appropriate modifications to incorporate the actual speed profiles that approaching drivers were observed to use in the field studies at intersections with various types of control.

It is recommended that the clear sight triangles used at locations where stopped vehicles must enter or cross a major road should be based on the concept of gap acceptance. Design values for use in the gap-acceptance model should be based on the actual gap-acceptance behavior of drivers observed in the field. The leg of the sight triangle along the minor road should be based on the actual vehicle stopping positions observed in the field.

The report presents sight distance criteria for intersections with each type of traffic control. Specific recommendations for intersection sight distance design policy are presented for consideration by the AASHTO Task Force on Geometric Design.

CHAPTER 1

INTRODUCTION

This report presents the results of research that included a comprehensive review of current geometric design policies for sight distance at intersections. The report presents new field data on traffic operations at intersections and proposes a revised geometric design policy for intersection sight distance (ISD).

This introduction presents an overview of intersection sight distance design policies; the NCHRP problem statement defining the need for this research; the research objectives and scope; and the organization of the remainder of the report.

OVERVIEW OF INTERSECTION SIGHT DISTANCE DESIGN POLICIES

The geometric design policies most widely used for highway design in the United States are those of the American Association of State Highway and Transportation Officials (AASHTO). The AASHTO policy on intersection sight distance design is presented on pages 696 through 721 of the 1994 AASHTO *Policy on Geometric Design of Highways and Streets* (also known as the AASHTO "Green Book") (*1*). The AASHTO Green Book considers intersection sight distance to be adequate when drivers at or approaching an intersection have an unobstructed view of the entire intersection and of sufficient lengths of the intersecting highways to permit the drivers to anticipate and avoid potential collisions. This requires unobstructed sight distance along both approaches of both intersecting roadways, as well as across their included corners (the so-called "clear sight triangles"). Adequate clear sight triangles are required both for drivers approaching an intersection where they may not be required to stop and for drivers who are stopped at an intersection to proceed safely to cross a major roadway or to turn left or right onto a major roadway. The 1994 Green Book policy has evolved from the previous policies presented in the 1984 and 1990 Green Books (*2,3*). This policy has now been presented for the first time in metric units in the 1994 Green Book.

The AASHTO policy provides geometric design criteria for five ISD cases, which are defined briefly in this section. One of the five cases has three subcases so there are, in effect, seven ISD cases. These seven cases are defined by the type of traffic control present at the intersection and the types of maneuvers that the drivers intend to make. The seven cases are as follows:

- Case I—ISD for vehicles approaching intersections with no control, at which vehicles are not required to stop, but may be required to adjust speed;
- Case II—ISD for vehicles on a minor-road approach controlled by a Yield sign;
- Case IIIA—ISD for a vehicle on a Stop-controlled approach on the minor road to accelerate from a stopped position and cross the major road;
- Case IIIB—ISD for a vehicle on a Stop-controlled approach on the minor road to accelerate from a stopped position and turn left into the major road;
- Case IIIC—ISD for a vehicle on a Stop-controlled approach on the minor road to accelerate from a stopped position and turn right into the major road;
- Case IV—ISD for a vehicle on a signal-controlled approach; and
- Case V—ISD for turning left from the major road at an intersection or driveway.

Tables 1 and 2 summarize the sight distances in AASHTO policy for each of these ISD cases in metric and English units, respectively, based on the 1994 and 1990 Green Books. Each of these seven cases is discussed in a later chapter of this report. Figure 1 illustrates the minimum sight triangles at intersections that should be kept clear of sight obstructions to implement the AASHTO ISD policy. Sight obstructions may include man-made structures, vegetation, trees, roadside appurtenances, and the terrain itself. Whether an obstacle or the terrain is a sight obstruction depends on not only the extent of the sight triangle, but also the horizontal and vertical alignment of the intersecting roads.

The various cases in the AASHTO ISD policy involve two different types of clear sight triangles. Drivers approaching uncontrolled and Yield-controlled intersections need sight distance before they reach the intersection so that they can decide whether to stop at the intersection or proceed. This requires *approach sight triangles,* like those shown in Figure 1 for ISD Cases I and II, that provide sight distance for drivers at a decision point as they approach the intersection. Drivers approaching an intersection on a Stop-controlled approach do not need a view of the intersecting roadway to decide whether to stop, because the presence of the Stop sign requires all vehicles to come to a stop before entering or crossing the major road. However, drivers stopped on the

TABLE 1 Summary of Current AASHTO ISD Design Policy (Metric Units) (*1*)

Design speed (km/h)	Intersection sight distance (m)						
	Case I	Case II[a]	Case IIIA[b]	Case IIIB	Case IIIC	Case IV	Case V[d]
20	20	–	–	–	–	–	–
30	25	30–30	40	65	65	[c]	35
40	35	45–45	50	90	90	[c]	40
50	40	60–65	55	120	120	[c]	50
60	50	75–85	65	160	160	[c]	55
70	60	95–115	75	205	205	[c]	65
80	65	115–140	85	255	255	[c]	70
90	75	135–170	95	310	310	[c]	80
100	85	160–205	100	380	380	[c]	85
110	90	180–250	110	455	455	[c]	95
120	100	205–290	120	–	–	[c]	100

[a] Equivalent to AASHTO SSD criteria.
[b] Based on crossing of a two-lane road by a passenger car.
[c] Use criteria for Cases IIIA, IIIB, and IIIC.
[d] Based on left turn from a two-lane road by a passenger car.

minor-road approach need sight distance along the major road to decide when it is safe to proceed. This requires *departure sight triangles* like those shown in Figure 1 for ISD Cases IIIA, IIIB, and IIIC.

To be fully appreciated, the AASHTO ISD policies must be understood in light of the other AASHTO sight distance requirements. Stopping sight distance (SSD) is the most universal type of sight distance. SSD must be provided continuously along every roadway, including locations on intersection approaches, to enable drivers to see objects and other vehicles in their path in sufficient time to stop their vehicle before reaching the object or conflicting vehicle. Thus, SSD is the fundamental geometric element which highway agencies use to ensure safe operations at every point on the highway system so that drivers who encounter a potential hazard can stop. SSD is currently measured from a driver eye height of 1,070 mm (3.50 ft) to an object height of 150 mm (6 in). Revisions to the AASHTO SSD policy have recently been recommended by Fambro et al. in "Determination of Stopping Sight Distances," the Final Report of a forthcoming NCHRP Project. These revisions are under consideration by the AASHTO Task Force on Geometric Design.

At intersections, SSD alone is not sufficient to ensure safe operations and ISD must be provided as well. SSD addresses only stationary objects (or vehicles) in the driver's path. ISD recognizes that, to ensure safe intersection operations, drivers also must see vehicles on intersecting approaches and vehicles starting from Stop signs. ISD differs from SSD in that (1) the targets that drivers must see are not always directly ahead in the roadway, but are often on intersecting roadways; (2) the targets that must be seen may be moving and may come into the driver's path; and (3) the decisions required of drivers often are more complex. On the other hand, in the case of ISD, the target that must be seen typically is another vehicle, so a target height much higher than the 150-mm (6-in) object used in SSD design is appropriate. The available sight distance on

TABLE 2 Summary of 1990 AASHTO ISD Design Policy (English Units) (*2*)

Design speed (mph)	Intersection sight distance (ft)						
	Case I	Case II[a]	Case IIIA[b]	Case IIIB	Case IIIC	Case IV	Case V[d]
10	45	–	–	–	–	–	–
15	70	–	–	–	–	–	–
20	90	125–125	210	230	230	[c]	170
25	110	150–150	260	290	290	[c]	210
30	130	200–200	300	380	380	[c]	250
35	155	225–250	350	470	470	[c]	290
40	180	275–325	400	580	580	[c]	330
45	200	325–400	430	700	700	[c]	370
50	220	400–475	490	840	840	[c]	410
55	240	450–550	540	990	990	[c]	450
60	260	525–650	590	1,160	1,160	[c]	490
65	285	550–725	630	1,360	1,360	[c]	530
70	310	625–850	680	1,590	1,590	[c]	570

[a] Equivalent to AASHTO SSD criteria.
[b] Based on crossing of a two-lane road by a passenger car.
[c] Use criteria for Cases IIIA, IIIB, and IIIC.
[d] Based on left turn from a two-lane road by a passenger car.

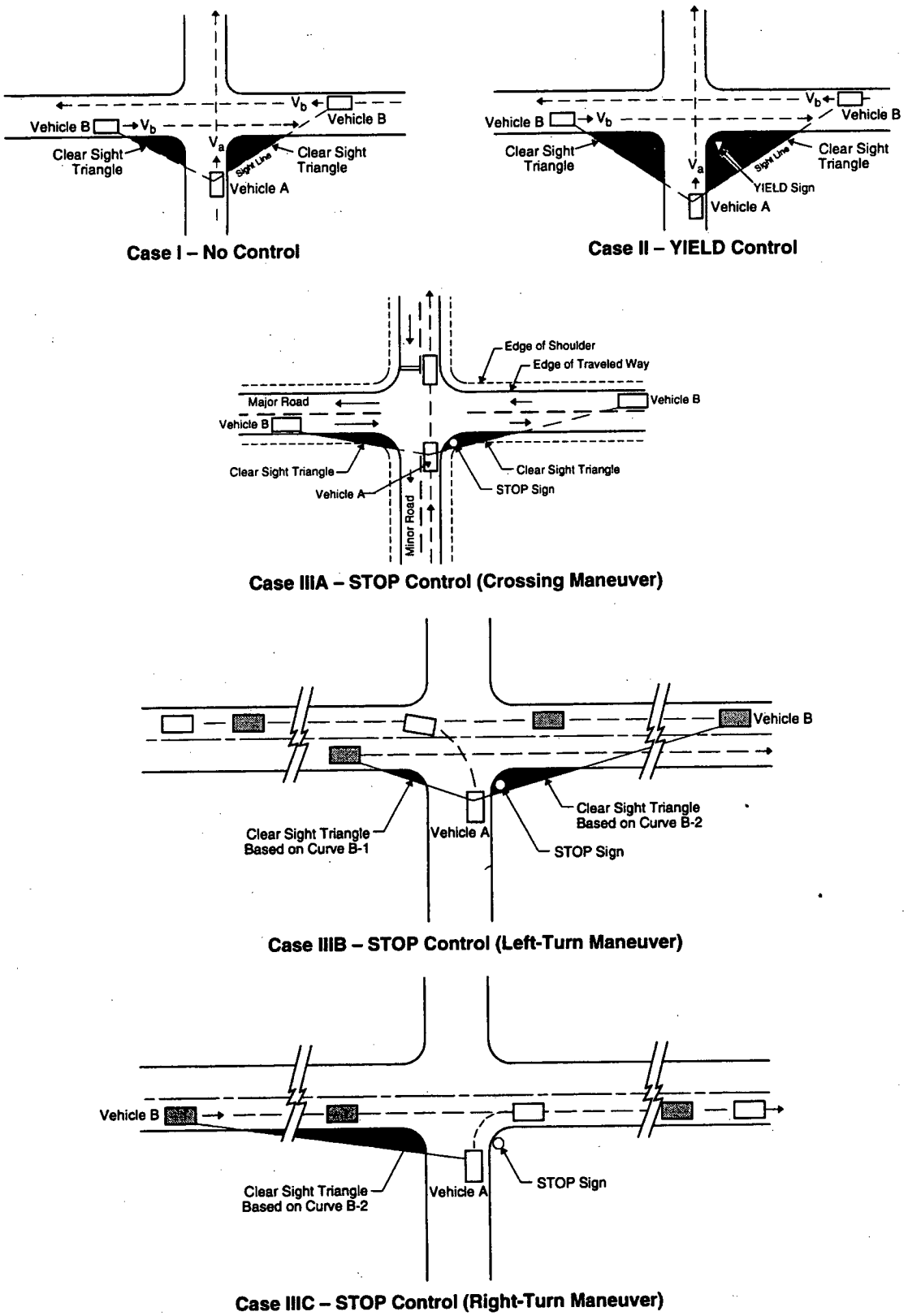


Figure 1. Examples of minimum clear sight triangles used for AASHTO ISD cases.

intersection approaches generally should be sufficient to permit each vehicle approaching an intersection to stop before reaching the intersection if a potentially conflicting vehicle is detected on an intersecting approach. In some cases, the available sight distance should be greater than just the minimum required to stop to promote desirable traffic operations at intersections and to ensure that vehicles with the legal right of way are not forced to stop unnecessarily.

The provision of the ISD is intended to give drivers an opportunity to obtain the information they need to decide whether to proceed, slow, or stop in situations where potentially conflicting vehicles may be present. ISD models and their associated parameter values should provide enough sight distance to enable most drivers in the most frequently encountered traffic situations to obtain the information that they need to make the correct decision about what action they should take. Since drivers are human beings with human failings, it is also desirable to provide a reasonable margin of safety to accommodate incorrect or delayed driver decisions. However, there are substantial costs associated with providing sight distance at intersections, so it is important that ISD requirements not be overly conservative or attempt to address traffic situations that are infrequent or unusual and for which increased ISD would provide little safety benefit.

ISD has an important role in minimizing intersection collisions, but it should be recognized that ISD alone cannot prevent such collisions because drivers may make incorrect decisions even at intersections where there are no sight restrictions.

RESEARCH PROBLEM STATEMENT

The NCHRP research problem statement for this research is presented below:

> Current procedures provided by the AASHTO *Policy on Geometric Design of Highways and Streets—1990* (the "Green Book") are intended to provide adequate sight distance at intersections to promote safe and efficient traffic operations. These procedures are applied to a variety of intersection traffic controls, including no control, Stop control, and signal control. The basic ISD models for these situations were formulated in 1940. Over the past 50 years, the model parameters have been modified to account for changes in the vehicle-driver-roadway system.
>
> Recently, questions have been raised about the validity of these models, as well as the appropriateness of certain parameter values used to calculate ISD. In addition, recent research has recommended possible alternative models, one of which includes the consideration of gap acceptance.
>
> It has been the experience of highway engineers applying the current AASHTO models that the ISD required for Cases IIIB and IIIC may be excessive. There also is concern that Case I ISD values may be unconservative. Moreover, situations such as a vehicle turning off the major roadway are not addressed. It is possible that these deficiencies may be contributing to misuse of ISD criteria by both state and local jurisdictions. Considering the high construction costs and uncertain safety benefits associated with long intersection sight distances, state highway officials have concluded that a substantial research effort is needed to reevaluate available information, collect additional data, and recommend improvements to current practice, if required.

RESEARCH OBJECTIVES AND SCOPE

The objective of this research was to evaluate the current AASHTO ISD methodology and, where appropriate, to recommend new or revised models and/or to recommend revised parameter values for use in those models. The research has addressed ISD Cases I, II, III, IV, and V.

Recommended ISD design procedures have been developed for each application based on a review of current and alternative practices, updated vehicle performance, roadway characteristics, and driver behavior data, including data collected in the field as part of the research. The research developed new concepts that provide a consistent framework for ISD models and used the field data that were collected to quantify the parameters of those models. Recommended revisions to ISD design policy were developed for consideration by the AASHTO Task Force on Geometric Design.

ORGANIZATION OF THIS REPORT

Chapters 2 through 5 of this report present the findings of the evaluation of ISD design policies for uncontrolled intersections, intersections with Stop control on the minor road, intersections with Yield control on the minor road, and intersections with traffic signal control, respectively. Each chapter addresses the current AASHTO policy, current state and local highway agency policies, identification of alternative ISD models and methodologies, evaluation of alternative ISD models and methodologies, and recommendations for the appropriate ISD case.

Chapter 6 presents the evaluation and recommendations for a variety of related ISD design issues including ISD policies for left turns from a major road, ISD policies for intersections with all-way Stop control, ISD policy for ramp terminals, the effect of intersection skew, the effect of the vertical profiles of intersection approaches, ISD measurement rules, and supplementary ISD policies for Stop-controlled intersections.

Chapter 7 presents the conclusions and recommendations of the research.

Appendix A presents the questionnaires used for a survey of state and local highway agencies concerning their ISD design policies that were conducted during the research. Appendix B summarizes the survey findings. Appendix C describes the ISD design policies that are used in selected foreign countries. Appendix D presents a model for the probability of vehicle-vehicle conflicts at uncontrolled intersections. Appendix E documents selected ISD models and methodologies that were considered as part of the research but did not become part of the research recommendations. Appendix F documents field studies of gap-acceptance behavior and accelerations and decelerations used by drivers at Stop-controlled intersections. Appendix G documents field studies of driver speed selection on approaches to uncontrolled and Yield-controlled intersections. Appendix H

documents the field studies of vehicle stopping positions on Stop-controlled approaches. Appendix I summarizes the investigation of tort liability issues related to ISD. Appendix J presents recommended revisions to the AASHTO Green Book for consideration by the AASHTO Task Force on Geometric Design.

Highway design activities in the United States currently are in transition from English to metric units. To aid readers in making this transition, the text of the report and most of the tables and figures use both systems of units. Key equations also are presented using both systems of units. Some older equations and design policies appear only in their original English units, and because the AASHTO Green Book is now entirely in metric units, the recommended revisions to the Green Book are presented only in metric units.

CHAPTER 2

EVALUATION OF ISD POLICY FOR INTERSECTIONS WITH NO CONTROL

This chapter presents an evaluation of the ISD policy for intersections with no Stop, Yield, or signal control on any of the approaches. Such intersections are addressed by ISD Case I in current AASHTO policy.

CURRENT AASHTO POLICY

The current AASHTO policy for ISD Case I addresses the sight-distance requirements for vehicles approaching intersections with no Stop signs, Yield signs, or traffic signals on any of the approaches. Figure 2 illustrates the clear sight triangle currently used by AASHTO to address this situation. AASHTO policy for ISD Case I is based on the assumption that drivers approaching an uncontrolled intersection do not necessarily need to stop, but do need sufficient time to adjust speeds to avoid colliding with one another.

AASHTO policy allows 3 sec on each approach for vehicles to adjust speeds. The AASHTO Green Book states the rationale for the 3-sec criterion as follows:

> A lower limit for the distance from an intersection, where a driver can first see a vehicle approaching on the intersecting road, is that which is traversed during 2.0 sec for perception and reaction plus an additional 1.0 sec to actuate braking or to accelerate to regulate speed (*1*).

Read literally, this might imply that the driver does not actually apply the brakes until 3 sec after first seeing the vehicle on an intersecting approach (2 sec for perception-reaction time and an additional 1 sec until the brakes are actuated). However, this interpretation does not appear reasonable, because at an intersection with minimum Case I ISD, neither driver would begin to brake or accelerate until he or she entered the intersection. In fact, the concept of perception-reaction time typically includes the time needed to begin taking the appropriate action (depressing the brake or accelerator pedal). Therefore, the current AASHTO policy for ISD Case I should be understood to permit 2 sec for perception-reaction time and 1 sec for acceleration or deceleration to adjust speed.

The following equation represents AASHTO's method to determine the minimum sight distance along each approach as presented in the 1984 and 1990 editions of the Green Book (*2,3*):

$$ISD = 1.47Vt \tag{1}$$

where:

ISD = d_a or d_b; minimum intersection sight distance (ft); (see Figure 2)
V = speed of vehicle (mph)
t = $t_{pr} + t_{adj}$ (sec) (assumed: t = 3.0 sec)
t_{pr} = perception-reaction time (sec) (assumed: t_{pr} = 2.0 sec)
t_{adj} = time required to regulate speed (sec) (assumed: t_{adj} = 1.0 sec)

The speed (V) in Equation (1) is generally assumed to be the design speed of the approach roadway, although this assumption is not stated explicitly in the Green Book.

Equation (2) presents a metric equivalent to Equation (1), based on the metric AASHTO policy that appears in the 1994 edition of the Green Book (*1*):

$$ISD = 0.278Vt \tag{2}$$

where the speed (V) is specified in kilometers per hour and the sight distance (ISD) is determined in meters.

Table 3 presents the ISD values from the AASHTO Green Book, which are calculated with Equations (1) and (2) and rounded to the nearest 5 or 10 ft or the nearest 5 m, in comparison to the current AASHTO stopping sight distance (SSD) values. The distances in Table 3 represent the sides of the approach sight triangle shown in Figure 2 and simply represent 3-sec travel time at the design speed of each approach. For example, if Highway A has a design speed of 80 km/h (50 mph) and Highway B has a design speed of 50 km/h (31 mph), then an unobstructed sight triangle with legs extending at least 65 m (213 ft) from the intersection along Highway A and 40 m (131 ft) from the intersection along Highway B would be recommended under current policy.

The sight distance values for ISD Case I presented in Table 3 are substantially less than the recommended AASHTO values for stopping sight distance (SSD). SSD is required at all locations on the highway system, but applies only to the

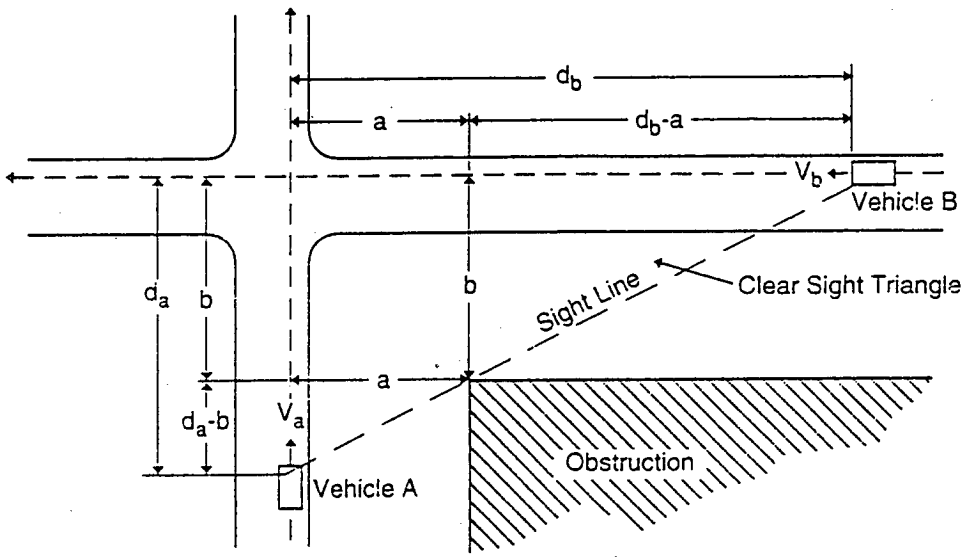


Figure 2. Minimum clear sight triangle used for ISD Case I.

view along the road on which the driver is traveling; ISD Case I applies not only to the view along that road, but also to the view across the corner to the intersecting road. This implies that a driver requires greater sight distance along the highway toward the intersection than across the corner toward potentially conflicting vehicles on an intersecting approach. Figure 3 compares the AASHTO sight distances for ISD Case I with the range of AASHTO SSD values.

Sight distance for ISD Case I is generally measured from a driver eye height of 1,070 mm (3.50 ft), the same as is currently used for SSD design, to an object height of 1,300 mm (4.25 ft), intended to represent the height of a potentially conflicting vehicle. In determining the requirements for a clear sight triangle, it is necessary not only to consider the position of roadside obstructions but also their height. Obstacles that are below or above the sight line may not be sight obstructions.

Clear sight triangles are needed in all four quadrants of a four-way uncontrolled intersection since potentially conflicting vehicles could arrive on any of the other approaches. Where the required sight triangle cannot be provided, the Green Book recommends that traffic control devices should be used to slow down or stop vehicles on one road, even if both roads are lightly traveled.

When two potentially conflicting vehicles approach an uncontrolled intersection at the same time, the AASHTO policy for ISD Case I is intended to allow the driver of each vehicle an opportunity to see the other and to adjust speed. According to the right-of-way rule that governs vehicle operations at uncontrolled intersections in most states, the vehicle approaching the intersection on the right has the right-of-way and the vehicle on the left must yield the right of way. However, the AASHTO policy for ISD Case I appears to assume that both vehicles may adjust their speeds, not just the vehicle required by law to yield.

The Green Book states that uncontrolled intersections should be used only in the design of rural intersections on lightly traveled two-lane roads where the cost of achieving

TABLE 3 AASHTO Criteria for ISD Case I in Comparison to SSD (*1*)

Speed (km/h)	Case I ISD (m)	Range of SSD[a] (m)	Speed (mph)	Case I ISD (ft)	Range of SSD (ft)
20	20	–	10	45	–
30	25	30–30	15	70	–
40	35	45–45	20	90	125–125
50	40	60–65	25	110	150–150
60	50	75–85	30	130	200–200
70	60	95–115	35	155	225–250
80	65	115–140	40	180	275–325
90	75	135–170	50	220	400–475
100	85	160–205	60	260	525–650
110	90	180–250	70	310	625–850
120	100	205–290			

[a] Rounded.

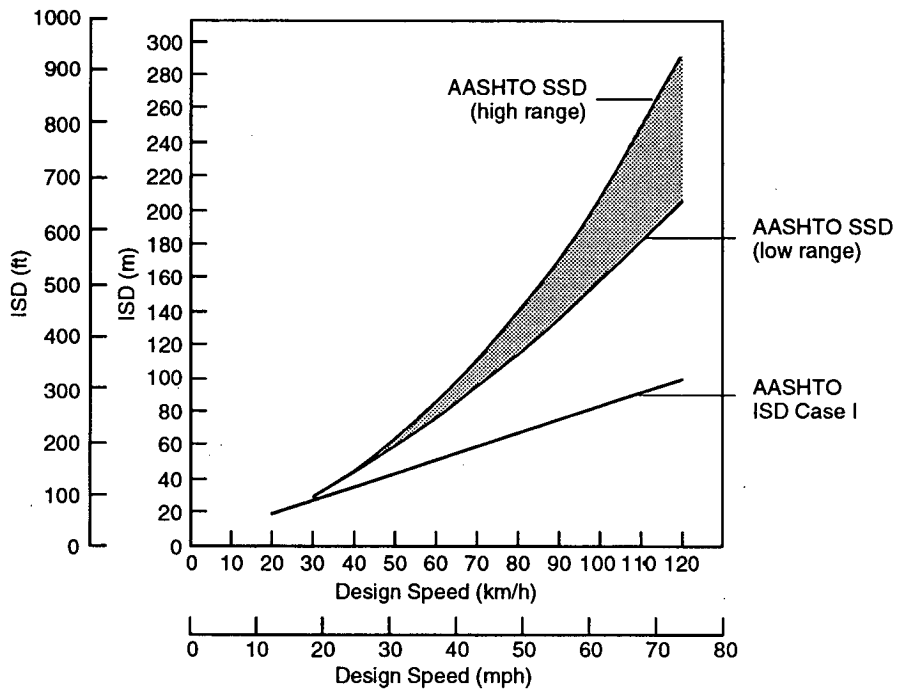


Figure 3. Comparison of AASHTO criteria for ISD Case I and SSD.

greater sight distance would be prohibitive (i.e., difficult to justify). In actual practice, uncontrolled intersections are also used for the intersections of low-volume local residential streets in urban and suburban areas, typically in newer subdivisions.

The Green Book states that provision of a sight triangle with a leg along each approach equal to SSD for that approach would be preferable to the criteria for ISD Case I. Then, a driver sighting a vehicle on an intersecting road would have the ability to stop, if necessary, before reaching the intersection. However, the use of SSD values as the basis for ISD is not required at uncontrolled intersections because the cost of providing additional sight distance is considered inappropriate for low-volume intersections.

CURRENT HIGHWAY AGENCY POLICIES

Table 4 summarizes the design policies used by state and local highway agencies for ISD at uncontrolled intersections. Nearly half of state highway agencies do not consider ISD Case I in their design policies because they do not permit uncontrolled intersections on the highway system under state jurisdiction. The table shows that, with only two exceptions, all state highway agencies that operate uncontrolled intersections have design policies based on ISD Case I from either the 1984 or 1990 Green Book. Furthermore, the 1984 and 1990 Green Book policies for AASHTO Case I are identical. It should be noted that the survey on which Table 4 is based (see Appendices A and B) was conducted before the publication of the 1994 Green Book.

Two state highway agencies have their own policies for ISD Case I that differ from AASHTO. These policies are as follows:

- One state has a policy for all ISD cases, including Case I, based on the sight distance values for AASHTO Case IIIA.
- One state uses a policy that is presented in Table 5 for *all* intersections and *all* sight distance cases. This policy provides
  - –SSD along the major road equal to twice the usual AASHTO SSD values;
  - –SSD along the minor road equal to the usual AASHTO SSD values; and
  - –A clear sight triangle with legs along the major and minor roads that equal or slightly exceed the AASHTO values for ISD Case I (uncontrolled intersections).

Table 4 shows that 46 percent of local agencies in urban areas and 35 percent of local agencies in rural areas use the AASHTO policy for ISD Case I. Approximately 32 percent of local highway agencies in urban areas have policies for ISD Case I that differ from AASHTO. Typical local sight-distance policies for uncontrolled intersections include:

- Specified clear sight triangles of various dimensions that apply specifically to uncontrolled intersections. For example, one local agency requires a sight triangle at uncontrolled intersections with legs equal to 26 m (85 ft) along the minor or entering street and 40 m (130 ft) along the major or cross street, measured from the potential point of impact for conflicting vehicles. This

TABLE 4 Summary of Highway Agency Design Policies for ISD Case I

Design policy	Number (percentage) of agencies					
	State highway agencies		Local agencies			
			Urban		Rural	
1990 AASHTO Green Book	15	(31.9)	12	(32.4)	6	(26.1)
1984 AASHTO Green Book	8	(17.0)	5	(13.5)	2	(8.7)
Own state policy	2	(4.3)	–	–	4[a]	(17.4)
Own local policy	–	–	12	(32.4)	2	(8.7)
Don't consider this case	22	(46.8)	8	(21.6)	9	(39.1)
Total	47		37		23	

[a] These local agencies use design policies for ISD Case I based on the policy of the state highway agency in their state.

policy is applicable to intersections of local, undivided, single-family residential streets with 40-km/h (25-mph) speed limits. If this sight-distance criterion is not satisfied, use of a Yield or Stop sign is required.

- Specified clear sight triangle based on a safe approach speed greater than 24 km/h (15 mph) determined from the safe-approach-speed procedure in the *Traffic Control Devices Handbook* (*4*).
- Specified clear sight triangle based on sight-distance requirements for a crossing maneuver similar to AASHTO Case IIIA.

Some local agencies that do not have a formal ISD policy for uncontrolled intersections have resurfacing, restoration, and rehabilitation (RRR) policies that state that neither ISD improvements nor installation of traffic control devices are required at existing uncontrolled intersections unless an accident problem develops. Other local agencies without a formal ISD policy that is specifically applicable to uncontrolled intersections rely on a clear sight triangle policy that is applicable to all intersections.

Finally, some local agencies in California (and possibly other states) use the AASHTO policy for ISD Case I, but

TABLE 5 Alternative ISD Policy Used By One State Highway Agency

Design speed (mph)	SSD along major road[a] (ft)	SSD along minor road[b] (ft)	Leg of clear sight triangle along major and minor roads[c] (ft)
30	400	200	160
35	450–500	225–250	160
40	550–650	275–325	185
45	650–800	325–400	200
50	800–950	400–475	220
60	1,050–1,300	525–650	260
70	1,250–1,700	625–850	310

[a] SSD along the major road is equal to twice the usual AASHTO SSD values.
[b] SSD along the minor road is identical to the usual AASHTO SSD values.
[c] Equivalent to distances d_a and d_b in Figure 2.

Note: This policy is used for all ISD Cases (I through IV).

assume an approach speed lower than the design speed of the roadway. Section 22352 of the California Vehicle Code specifies a prima facie speed limit of 24 km/h (15 mph) for drivers under several conditions including:

> [W]hen traversing any intersection of highways, if during the last 100 ft of the driver's approach to the intersection, the driver does not have a clear and unobstructed view of the intersection and of any traffic along all of the highways entering the intersection for distance of 100 ft along all those highways, except at an intersection protected by stop signs or yield right-of-way signs or controlled by official traffic control signals. (*5*)

Based on this legal provision, some local agencies assume an approach speed of 24 km/h (15 mph) when applying the AASHTO criteria for ISD Case I.

ASSESSMENT OF CURRENT POLICIES

The following discussion presents an assessment of the current AASHTO, state, and local policies for ISD Case I.

AASHTO Policy

Time to Adjust Speed

An analysis of ISD Case I indicates that the current AASHTO model, with its current assumptions about driver behavior in approaches to uncontrolled intersections, does not necessarily provide enough time for vehicles to adjust speeds to avoid a collision. For an uncontrolled intersection with minimum Case I sight distance, the most critical condition arises when two vehicles traveling at the design speed of their respective approach roadways arrive simultaneously on intersecting approaches at locations equivalent to a travel time of 3 sec from the intersection. Our analysis indicates that, under the current AASHTO assumptions, 1 sec is not sufficient time for drivers to adjust speed, even if both drivers choose correct responses (one driver traveling at constant speed or accelerating and the other driver braking).

The conclusion stated above can be illustrated with a numerical example. Figure 2 illustrates an uncontrolled intersection with two vehicles (A and B) approaching. In the following example, we will assume that both vehicles arrive simultaneously at a distance from the intersection equivalent to 3-sec travel time at the appropriate design speed. For illustrative purposes, we will assume that both roadways are 7.3 m (24 ft) wide and have the same design speed, although the same computations could be performed for any combination of design speeds. The following assumptions are made:

- Vehicle A travels at a constant speed equal to the design speed during 2 sec of perception-reaction time.
- After 2 sec, Vehicle A begins to decelerate at a rate equivalent to the braking coefficients assumed in the AASHTO stopping sight distance criteria (0.40 at 30 km/h or 20 mph, decreasing to 0.28 at 120 km/h or 70 mph). These braking coefficients are used to define the deceleration rate of Vehicle A. The braking coefficients used in stopping sight distance approach the maximum deceleration that can be expected from a vehicle with poor tires on a poor, wet pavement; however, they represent comfortable deceleration rates on a dry pavement and are much less than the maximum deceleration rate that can be achieved.
- Vehicle B is assumed to travel at a constant speed equal to the design speed until it clears the intersection. Vehicle B neither accelerates nor decelerates. The intersection is assumed to be 7.3 m (24 ft) wide and Vehicle B is assumed to be a passenger car 5.8 m (19 ft) in length (based on the length of the AASHTO passenger car design vehicle).
- This scenario is evaluated for Vehicle A approaching the intersection to the left of Vehicle B, as shown in Figure 2, and for Vehicle A approaching the intersection to the right of Vehicle B, the opposite of the configuration shown in Figure 2. The situation with the decelerating vehicle (Vehicle A) on the left is in accordance with the legal requirements of the right-of-way rule. The situation with Vehicle A on the right is opposite to the requirements of the right-of-way rule.

The results of the analyses of the two scenarios described above are summarized in the following tables:

- Table 6—Vehicle B approaching from the right at constant speed, and
- Table 7—Vehicle B approaching from the left at constant speed.

Tables 6 and 7 present the time required for Vehicle A to reach the center of the intersection in comparison to the time required for Vehicle B to clear (pass completely through) the center of the intersection. The difference between the time for Vehicle B to clear the center of the intersection and the time for Vehicle A to reach the center of the intersection is referred to as the margin of safety.

It should be noted that for most of the situations addressed by Tables 6 and 7, the time margin of safety is negative. This implies that the approaching vehicles cannot avoid collision unless (1) one or both of the vehicles accelerate or brake at unusually high rates or (2) one or both of the vehicles make a major change in path (e.g., encroaching on an adjoining lane or shoulder) to avoid collision.

It is also interesting to note that the margins of safety in Table 6, while slightly less than zero, are less critical than the margins of safety in Table 7. This illustrates the engineering rationale for the right-of-way rule. Given the intersection geometry that

TABLE 6 Margin of Safety for an Uncontrolled Intersection Designed in Accordance with AASHTO Case I (Vehicle B Approaching from the Right at Constant Speed)

Design speed of both roadways, km/h (mph):	32 (20)	40 (25)	48 (30)	56 (35)	64 (40)	72 (45)	80 (50)	88 (55)	97 (60)	105 (65)	113 (70)
Vehicles A and B											
Travel time (sec) to intersection at beginning of maneuver[a]	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Distance from intersection at beginning of maneuver, m (ft)[a]	27 (88)	34 (110)	40 (132)	47 (154)	54 (176)	60 (198)	67 (220)	74 (242)	81 (264)	87 (286)	94 (308)
Vehicle A											
Time (sec) to reach center of intersection[b]	–[c]	4.0	3.6	3.5	3.4	3.3	3.3	3.2	3.2	3.2	3.2
Vehicle B											
Clearance distance, m (ft)[d]	36 (119)	43 (141)	50 (163)	56 (185)	63 (207)	70 (229)	77 (251)	83 (273)	90 (295)	97 (317)	103 (339)
Clearance time (sec)[e]	4.1	3.8	3.7	3.6	3.5	3.5	3.4	3.4	3.4	3.3	3.3
Margin of safety between Vehicles A and B											
Time margin of safety (sec)[f]	–[c]	+0.2	–0.1	–0.1	–0.1	–0.2	–0.1	–0.2	–0.2	–0.1	–0.1

[a] Based on travel at constant speed equal to the design speed, per AASHTO Case I.
[b] Vehicle A will not collide with Vehicle B until Vehicle A crosses the center of the intersection.
[c] Vehicle A can stop before reaching the center of the intersection.
[d] Based on distance from intersection plus half of intersection width (3.7 m or 12 ft) plus vehicle length (5.8 m or 19 ft).
[e] Based on constant speed travel at the design speed.
[f] Clearance time for Vehicle B minus time for Vehicle A to reach center of intersection.

results from vehicles driving on the right side of each roadway, the margins of safety (and, thus, the sight distance requirements) are less critical if the vehicle on the left is required to yield than if the vehicle on the right is required to yield.

The results presented in Table 6 raise a definite concern that, if the assumptions of the current AASHTO model are correct, there could be a potential for accidents at intersections designed in accordance with the existing AASHTO Case I. However, it must be noted that the accident potential at such intersections is limited by the traffic volumes which typically are very low; the next section addresses the probability of collisions at low-volume, uncontrolled intersections.

Another concern is that, if both Vehicles A and B brake upon seeing one another, this can result in a situation that is more critical than the scenario addressed above in Table 6. Since there may not be enough distance for either vehicle to stop (AASHTO Case I ISD is less than SSD), braking by both vehicles makes a collision more likely than if one of the vehicles continues to travel at the design speed. However, this situation is a potential concern because braking by both vehicles seems a reasonably likely response of drivers who encounter one another at an uncontrolled intersection. While this concern is mentioned in the AASHTO Green Book, it is not addressed by the current criteria for ISD Case I.

Finally, it must be recognized that the existing AASHTO Case I model, and all of the scenarios discussed above, assume that both vehicles proceed at the design speed of their respective approach roadways until they are within 1.0 sec of reaching the intersection. Even casual observation suggests that vehicles approaching uncontrolled intersections do slow substantially, even when potentially conflicting vehicles are not present. The subsequent evaluation of alternative ISD models and methodologies includes a field study of approaches to uncontrolled intersections to document observed driver behavior and to evaluate how the observed driver behavior should be incorporated in ISD design policy.

Probability of Collisions at Uncontrolled Intersections

Part of the rationale for the 3-sec ISD allowance for uncontrolled intersections is that most uncontrolled intersections have very low traffic volumes, so that vehicle-vehicle conflicts are rare. An analysis of the expected number of vehicle-vehicle conflicts per day at different traffic-volume levels has been performed to assess the potential for collisions at uncontrolled intersections. The starting point for this

TABLE 7 Margin of Safety for an Uncontrolled Intersection Designed in Accordance with AASHTO Case I (Vehicle B Approaching from the Left at Constant Speed)

Design speed of both roadways, km/h (mph):	32 (20)	40 (25)	48 (30)	56 (35)	64 (40)	72 (45)	80 (50)	88 (55)	97 (60)	105 (65)	113 (70)
Vehicles A and B											
Travel time (sec) to intersection at beginning of maneuver[a]	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Distance from intersection at beginning of maneuver, m (ft)[a]	27 (88)	34 (110)	40 (132)	47 (154)	54 (176)	60 (198)	67 (220)	74 (242)	81 (264)	87 (286)	94 (308)
Vehicle A											
Time (sec) to enter intersection[b]	3.5	3.3	3.2	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.0
Vehicle B											
Clearance distance, m (ft)[c]	40 (131)	47 (153)	53 (175)	60 (197)	67 (219)	74 (241)	80 (263)	87 (285)	94 (307)	100 (329)	107 (351)
Clearance time (sec)[d]	4.5	4.2	4.0	3.8	3.7	3.7	3.6	3.5	3.5	3.5	3.4
Margin of safety between Vehicles A and B											
Time margin of safety (sec)[e]	–1.0	–0.9	–0.8	–0.7	–0.6	–0.6	–0.5	–0.4	–0.4	–0.4	–0.4

[a] Based on travel at constant speed equal to the design speed, per AASHTO Case I.
[b] Vehicle A will not collide with Vehicle B until Vehicle A enters the intersection.
[c] Based on distance from intersection plus intersection width (7.3 m or 24 ft) plus vehicle length (5.8 m or 19 ft).
[d] Based on constant speed travel at the design speed.
[e] Clearance time for Vehicle B minus time for Vehicle A to enter the intersection.

analysis was the estimation technique formulated by Glennon (*6*) for *NCHRP Report 214: Design and Traffic Control Guidelines for Low-Volume Rural Roads.* This technique was based on a uniform arrival distribution for traffic over an 18-hr day. We have generalized this technique into a Poisson arrival model that allows the computation of expected conflict frequencies as a function of the approaching traffic volumes and the conflict-exposure interval (the minimum time between vehicle arrivals on intersecting approaches that are considered to be in conflict with one another). For example, at an uncontrolled intersection of two low-volume roads or streets, each with an average daily traffic (ADT) of 100 veh/day, it would be expected that two vehicles on intersecting approaches would arrive within 2 sec of one another an average of 0.31 times per day (113 times per year). The expected conflict frequency would increase to 4.91 times per day (1,800 times per year) for traffic volumes of 400 veh/day on both roads. Arrival times less than 2 sec apart for vehicles on intersecting approaches are less likely, but would produce greater accident potential. This Poisson model is presented in Appendix D of this report.

The Poisson model was also used to determine the probability of conflicting vehicle arrivals within a shorter time period of one another, 0.6 sec. At the intersection of two roads, each with an ADT of 100 veh/day, conflicting arrivals within 0.6 sec of one another would be expected at the rate of 0.09 times per day (33 times per year). For the intersection of roads with an ADT of 400 veh/day, the frequency of these more severe conflicts would increase to 1.48 times per day (540 times per year).

The AASHTO Green Book recognizes that the criteria for ISD Case I are not necessarily adequate to prevent collisions when an approaching vehicle encounters a succession of two or more vehicles (i.e., a platoon) on an intersecting approach. The Poisson model in Appendix D can be used to establish that, on low-volume roads, the simultaneous arrival of a vehicle on one approach and a platoon on another approach is, indeed, a rare event. At the intersection of two roads, each with an ADT of 100 veh/day, one would expect an average of 0.00024 vehicle-platoon conflicts per day (or about one conflict every 11 years). The expected number of vehicle-platoon conflicts increases to 0.01515 per day (or about 5.5 conflicts per year) for the intersection of roads with ADTs of 400 veh/day. This analysis is described further in Appendix D.

Our assessment of the results obtained with the Poisson arrival model is that, even at ADTs as low as 100 veh/day, vehicle-vehicle conflicts at low-volume uncontrolled intersections do potentially occur with sufficient frequency that

the design of such intersections should be based on a model that ensures adequate sight distance for safe operations. In other words, the frequencies of potential vehicle-vehicle conflicts are not so low that concern about sight distance at low-volume intersections can be dismissed out of hand. On the other hand, this does not necessarily indicate that the sight distance requirements for ISD Case I must necessarily be increased; alternative ISD models based on observed real-world driver behavior at uncontrolled intersections should be considered. By contrast, conflicts between a vehicle on one approach and a platoon of two or more vehicles on an intersecting approach do appear to occur so infrequently that multiple-vehicle platoons need not be considered in formulating an ISD model for uncontrolled intersections.

Perception-Reaction Time

The current AASHTO policy for ISD Case I assumes that the driver of Vehicle A approaching an uncontrolled intersection will be able to detect Vehicle B on the crossroad and react to that vehicle within 2 sec. Given the speeds and positions of the vehicles involved, this means that Vehicle B on the crossroad will be out of the central part of the visual system of the driver in the approaching Vehicle A. For example, if the design speeds for both roads are 40 km/h (25 mph), then both vehicles will be 33 m (108 ft) from the intersection at the beginning of the 3-sec interval allowed by AASHTO policy for perception-reaction time and adjusting speed. This would put Vehicle B on the crossroad at the edge of a 90-degree visual arc for the driver of the approaching vehicle. While this is well outside the central part of the visual system, it does fall within the accepted ranges for peripheral vision. However, if the design speed for the crossroad is 80 km/h (50 mph) and the design for the approach road is 40 km/h (25 mph), then Vehicle B on the crossroad will be 67 m (220 ft) from the intersection and the approaching Vehicle A will be 33 m (108 ft) from the intersection at the start of the 2-sec perception-reaction time. This would put Vehicle B at about 63.4 degrees to the left (or right) of Vehicle A's direction of travel. Thus, a total visual arc of 127 degrees would be needed for the driver of Vehicle A, which is near the limit of the peripheral vision system for some drivers. This scenario could be especially critical for drivers who experience a lessening of sensitivity in the peripheral vision system with age.

Of course, these scenarios do not account for head movements on the part of the driver of Vehicle A. However, at nonintersection locations, most of the primary visual scanning of the road is done with eye movements. Head movements may play a more important role in detecting potentially conflicting vehicles once the driver becomes aware that there is an intersection ahead.

Another consideration related to perception-reaction time is that there is greater sensitivity to motion in the peripheral vision system than there is to recognition of form. This does not mean that peripheral vision is more sensitive to motion than the central part of the visual system. The eye is less sensitive to both motion and form in the periphery, but the sensitivity to motion decreases less rapidly than sensitivity to form. Therefore, drivers are much more likely to detect a moving object in their periphery than to recognize the moving object as a vehicle.

Still another consideration is whether the driver is in an alerted or an unalerted condition when approaching an uncontrolled intersection. If the horizontal and vertical geometry are such that a driver can recognize that there is an intersection ahead, even if there are no vehicles on the crossroad, then the driver will be in an alerted condition and his or her visual scanning and head movements will be different than if the driver is not aware of the intersection ahead. The same would be true of drivers who are familiar with the road and the location of the intersections along that road. However, it is also true that unfamiliar drivers may often be unaware that the intersection is, in fact, uncontrolled and may assume that they have the right of way. It usually is difficult to tell from a distance whether there is a Stop or Yield sign present on the intersecting approach.

The combination of vehicles present at the vertices of the sight triangle and the drivers being in an unalerted condition represents something of a worst case scenario for perception-reaction time. Unfortunately, this scenario seems more likely to occur at an uncontrolled intersection than for the other ISD cases.

No empirically based study of perception-reaction time related to driving has used stimuli that are outside of what could be classified as the central visual system. One study of eye movement latency tested targets at the edge of an 80-degree total visual arc, but this may be the most extreme case that has been investigated.

One study of perception-reaction times for Case I by McGee and Hooper (*7*) used an approach wherein estimates were assembled for the constituent parts of the perception-reaction process. These estimates were then summed to determine an appropriate perception-reaction time. The constituent parts of the perception-reaction process considered by McGee and Hooper were

1. Driver picks up (through peripheral vision) an object moving toward the intersection;
2. After a latency period, eye or head movement or both detect the object;
3. Object is recognized as a vehicle;
4. Opposing vehicle's speed and time to reach intersection are estimated;
5. Decision is made on whether deceleration or acceleration is required; and
6. Decided action is initiated (that is, foot moves to brake pedal).

McGee and Hooper recommend perception-reaction times equal to 2.6 sec for the 50th percentile of the driving population, 3.4 sec for the 85th percentile, and 4.0 sec for the 95th

percentile. The potential problem with these recommendations is that simple addition of the components of the perception-reaction process does not allow for time overlaps or parallel processing of these components. In addition, it may also be unreasonable to assume that the 85th-percentile driver for one component of perception-reaction time will also be the 85th-percentile driver for the other components.

The authors of this report agree with the assessment that the perception-reaction requirements of ISD Case I are potentially greater than the other ISD cases because, since no traffic control devices are present, drivers approaching an uncontrolled intersection are less likely to be aware of the presence of the intersection than they would if Stop or Yield signs were present. Thus, based on the greater likelihood of an unalerted condition, we recommend a perception-reaction time for Case I that is greater than the 2.0-sec value used in Case III. However, we also consider that McGee and Hooper have been too conservative in combining the components of the perception-reaction process in additive fashion. McGee and Hooper recommend a perception-reaction value of 2.6 sec as appropriate for the 50th percentile of the driving population. In our judgment, this value (rounded for design purposes to 2.5 sec) is appropriate for a much higher percentage of the driving population. Thus, we recommend a perception-reaction time for ISD Case I of 2.5 sec, which is identical to the value recommended for use in SSD design. This is appropriate because, while the perceptual requirements of SSD and ISD Case I differ, both represent unalerted conditions.

Other State and Local Policies

The ISD policy used by one state presented in Table 5 compares favorably to the AASHTO policy for ISD Case I. The legs of the clear sight triangle in Table 5 are the same as or longer than those in AASHTO Case I and the provision of above-minimum SSD along the major road reduces the possibility that vertical or horizontal curvature will contribute to restricted ISD. Of course, at a four-leg uncontrolled intersection the distinction in SSD requirements between the major and minor roads shown in Table 5 is difficult to make.

Another state agency applies the ISD requirements for AASHTO Case IIIA to all intersections. The sight distances along the major road assumed by Case IIIA are longer than the sight distances for Case I, so this policy is conservative if clear sight triangles defined by the Case IIIA sight distances are provided along all legs of the intersection. However, it should be noted that the Case IIIA sight distances were derived for crossing of the major road by a stopped vehicle on a Stop-controlled minor-road approach and were not intended for application to uncontrolled intersections.

Alternative methodologies for ISD Case I, including methodologies based on current highway agency practices, are addressed in the next section.

ALTERNATIVE ISD MODELS AND METHODOLOGIES

The following candidate ISD methodologies for uncontrolled intersections were identified and considered in the early stages of the research as alternatives to the current AASHTO policy:

- Current AASHTO model with increased time to adjust speed.
- Current AASHTO model with increased perception-reaction time.
- Current AASHTO model assuming approach speeds lower than the design speed, if justified by actual driver behavior in the field.
- Alternative model that provides sufficient ISD for approaching vehicles to stop, if necessary, before reaching the intersection.
- Alternative model developed by McGee et al. (*8*) that explicitly incorporates deceleration rate.
- Alternative model developed by Harwood et al. (*9*) that explicitly incorporates deceleration rate and vehicle length.
- Alternative model based on safe approach speed, such as that presented in Figure 2-5 in the *Traffic Control Devices Handbook* (*4*), or a modification of it.

A decision was reached based on the margins of safety presented in Tables 6 and 7 that a change in the current AASHTO model for ISD Case I was necessary. As explained above, a decision was also reached that it was appropriate to increase the perception-reaction time for drivers on approaches to uncontrolled intersections from 2.0 to 2.5 sec.

The alternative models developed by McGee et al. (*8*) and Harwood et al. (*9*) and presented in the *Traffic Control Devices Handbook* (*4*) were evaluated and drawbacks were found in each model that appeared to make them inappropriate for use in AASHTO policy. Appendix E provides an explanation of the reasons why these models were not considered further.

Thus, the development of a recommended sight-distance design policy for uncontrolled intersections came down to an evaluation of two key issues. These issues were: (1) whether the ISD model for uncontrolled intersection approaches should be based on adjusting speed or stopping; and (2) whether an approach speed lower than the design speed or midblock running speed of the approach roadway could be assumed. These issues are considered in the remainder of this chapter.

EVALUATION OF ALTERNATIVE ISD MODELS AND METHODOLOGIES

An evaluation was conducted of alternative ISD models and methodologies for approaches to uncontrolled intersections based on adjusting speed and on stopping.

Alternative Model Based on Increased Time to Adjust Speed

One option for increasing the margin of safety for vehicles approaching uncontrolled intersections is to increase the time available to adjust speeds [t_{adj} in Equations (1) and (2)]. Figure 4 compares the current AASHTO criteria for Case I, which allows 1.0 sec to adjust speeds, with revised criteria based on 1.5, 2.0, 2.5, and 3.0 sec to adjust speeds. The figure shows that, for a vehicle speed of 80 km/h (50 mph), the appropriate ISD value increases from 67 m (220 ft), when 1.0 sec is allowed to adjust speed, to 111 m (364 ft), when 3.0 sec is allowed to adjust speed. Figure 5 shows comparable criteria when the perception-reaction time is increased from 2.0 to 2.5 sec, as well as increasing the time allowed to adjust speed. In that case, the required ISD increases from 78 m (255 ft), when 1.0 sec is allowed to adjust speed, to 122 m (400 ft), when 3.0 sec is allowed to adjust speed.

A sensitivity analysis was conducted to determine the value of time to adjust speed at which all of the margins of safety shown in Table 6 would become nonnegative. The result is illustrated in Table 8 which shows that all values of the margin of safety become nonnegative when the time to adjust speed reaches 1.6 sec. In practice, the use of a value of t_{adj} somewhat higher than 1.6 sec would be prudent to allow for the possibility that drivers will delay making a decision or make the wrong decision. Therefore, a design value for time to adjust speed of 2.0 sec is suggested.

Table 9 presents candidate ISD criteria for a range of design speeds based on the assumption of 2.5 sec for perception-reaction time and 2.0 sec to adjust speed. The table shows that these assumptions result in substantially greater sight-distance values than the current AASHTO criteria.

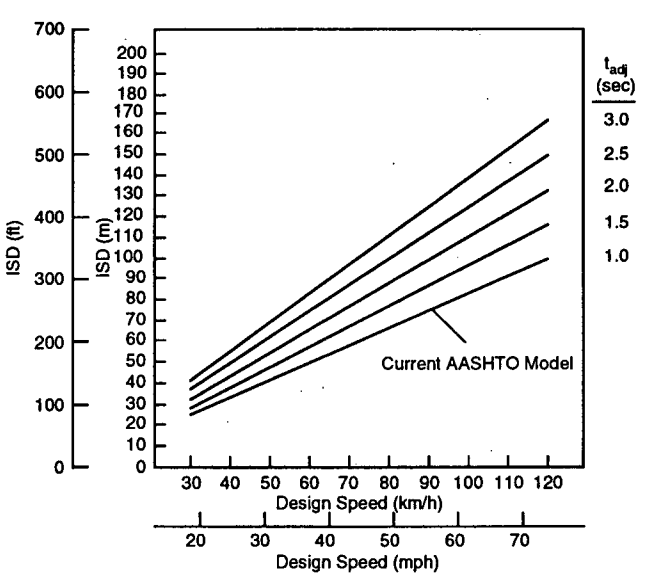


Figure 4. Effect of changes in time to adjust speed on ISD for Case I.

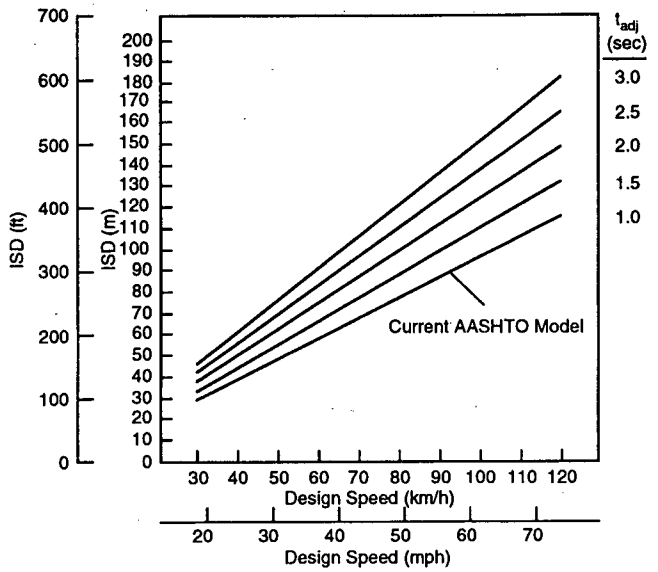


Figure 5. Effect of changes in time to adjust speed on ISD for Case I with perception-reaction time increased to 2.5 sec.

It should be noted in Table 8 that, when a constant time to adjust speed of 1.6 sec is assumed, the calculated margin of safety is higher at lower design speeds than at higher design speeds. This suggests the possibility of allowing the time to adjust speed to increase with increasing design speed to maintain a constant margin of safety across all design speeds. This concept makes sense and would probably have been recommended except that, as explained below, an alternative sight-distance model based on stopping rather than adjusting speed was found to be preferable for uncontrolled intersections.

Alternative Model Based on Vehicle Stopping

Another alternative model considered is based on providing sufficient sight distance for the vehicles on each approach to stop before they reach the intersection. This could be achieved by providing a clear sight triangle with legs equivalent to SSD in each quadrant of the intersection. The question remains, however, as to what approach speed should be used to determine SSD. A field study conducted to evaluate driver speed behavior on approaches to uncontrolled intersections is described below.

Field Study Results

A field study of driver behavior on approaches to uncontrolled intersections was conducted on approaches to seven uncontrolled intersections in Phoenix, Arizona. All of the field studies were conducted on lower-volume residential streets with average midblock speeds in the range of 35 to

TABLE 8 Margin of Safety for an Uncontrolled Intersection Designed With 1.6 Sec for Vehicle A to Adjust Speed (Vehicle B Approaching from the Right at Constant Speed)

Design speed of both roadways, km/h (mph):	32 (20)	40 (25)	48 (30)	56 (35)	64 (40)	72 (45)	80 (50)	88 (55)	97 (60)	105 (65)	113 (70)
Vehicles A and B											
Travel time (sec) to intersection at beginning of maneuver[a]	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
Distance from intersection at beginning of maneuver, m (ft)[a]	32 (106)	40 (132)	48 (158)	56 (185)	64 (211)	73 (238)	81 (264)	88 (290)	97 (317)	105 (343)	113 (370)
Vehicle A											
Time (sec) to reach center of intersection[b]	–[c]	–[c]	5.1	4.5	4.2	4.1	4.0	4.0	3.9	3.9	3.9
Vehicle B											
Clearance distance, m (ft)[d]	42 (137)	50 (163)	58 (189)	66 (216)	74 (242)	82 (269)	90 (295)	98 (321)	106 (348)	114 (374)	122 (401)
Clearance time (sec)[e]	4.7	4.4	4.3	4.2	4.1	4.1	4.0	4.0	4.0	3.9	3.9
Margin of safety between Vehicles A and B											
Time margin of safety (sec)[f]	–[c]	–[c]	+0.8	+0.3	+0.1	+0.1	0.0	0.0	0.0	0.0	0.0

[a] Based on travel at constant speed equal to the design speed, per AASHTO Case I.
[b] Vehicle A will not collide with Vehicle B until Vehicle A crosses the center of the intersection.
[c] Vehicle A can stop before reaching the center of the intersection.
[d] Based on distance from intersection plus half of intersection width (3.7 m or 12 ft) plus vehicle length (5.8 m or 19 ft).
[e] Based on constant speed travel at the design speed.
[f] Clearance time for Vehicle B minus time for Vehicle A to reach center of intersection.

48 km/h (22 to 30 mph) and 85th percentile midblock speeds in the range of 42 to 52 km/h (26 to 32 mph). The overall average midblock speed was 40 km/h (25 mph) and the overall 85th percentile midblock speed was 48 km/h (30 mph). Higher speed rural intersections were not included because none of the candidate locations identified had sufficient traffic volume for a productive field study.

The field study determined the speed profiles of vehicles that approached an uncontrolled intersection and went straight through the intersection (without turning right or left) when no potentially conflicting traffic or pedestrians were present. A total of 226 such vehicles were observed and their speed profiles were recorded using laser speed guns. Speed data were recorded at two locations: (1) at a midblock location, typically 120 to 180 m (400 to 600 ft) in advance of the intersection; and (2) on the intersection approach, typically within the last 60 m (200 ft) before vehicle reached the intersection.

The field study found that 23 percent of the approaching drivers of through vehicles stopped at the intersection even though there were no conflicting vehicles present, there were no other vehicles present, and there was no legal requirement to stop. The average midblock speed was approximately 40 km/h (25 mph) and the average minimum speed on the intersection approach was approximately 18 km/h (11 mph). The average speed reduction from the midblock location to the intersection approach was 19 km/h (12 mph), or 48 percent of the midblock speed. Only 6 percent of drivers traveled through the intersection at constant speed or increased their speed on the intersection approach. This finding suggests that it is reasonable to assume that drivers approaching uncontrolled intersections on urban and suburban residential streets typically slow to about half of their midblock running speeds.

The field data show that the deceleration rates used in slowing down on uncontrolled intersection approaches are very gradual, typically 1.5 m/sec^2 (5.0 ft/sec^2) or less.

The field study results are presented in greater detail in Appendix G of this report.

Interpretation of Field Study Results

The field study results suggest that the current AASHTO model is very conservative if it is assumed that drivers approaching uncontrolled intersections travel at a constant speed equal to the midblock design speed unless they encounter a potentially conflicting vehicle. The field data show that drivers slow for an uncontrolled intersection, whether a potentially conflicting vehicle is in view or not. Such drivers are then in a position to brake to a stop from a speed slower than their midblock speed if a potentially conflicting vehicle comes into view.

TABLE 9 Potential ISD Criteria for Uncontrolled Intersections Based on Increased Time to Adjust Speed

Design speed (km/h)	Sight distance (m) Based on increased time to adjust speed[a]	Sight distance (m) Based on current AASHTO assumptions[b]	Design speed (mph)	Sight distance (ft) Based on increased time to adjust speed[a]	Sight distance (ft) Based on current AASHTO assumptions[b]
20	25	20	10	70	45
30	40	25	15	100	70
40	50	35	20	135	90
50	65	45	25	165	110
60	75	50	30	200	130
80	100	70	35	240	155
100	125	85	40	270	180
110	140	95	50	330	220
			60	400	260
			70	470	310

[a] Based on 2.5 sec for perception-reaction time and 2.0 sec to adjust speed.
[b] Based on 2.0 sec for perception-reaction time and 1.0 sec to adjust speed.

The field study results suggest that a speed profile model like that shown in Figure 6 presents typical driver behavior on an uncontrolled intersection approach. Because drivers slow down even when no potentially conflicting vehicles are present, it appears reasonable to assume that, if two vehicles are approaching an uncontrolled intersection on intersection approaches, both vehicles would slow down in accordance with the speed profile shown in Figure 6. Then, when the drivers see one another, the driver without the right of way could brake (to a stop, if necessary, as shown by the dashed line in the speed profile in Figure 7) and the driver with the right of way could proceed without further braking. A conservative ISD policy for uncontrolled intersections would allow for the possibility that both drivers might choose to brake to a stop and allow sufficient sight distance for them to do so without colliding. However, as shown in Figure 7, it can be assumed that braking to a stop would begin at a speed less than the midblock running speed.

ISD Model Based on Stopping from a Reduced Approach Speed

An ISD model has been formulated based on the speed profile illustrated in Figure 7. The assumptions of this ISD model are as follows:

- A vehicle approaching an uncontrolled intersection will decelerate as it nears the intersection whether a potentially conflicting vehicle can be seen on an intersecting approach or not. This deceleration will be at a gradual

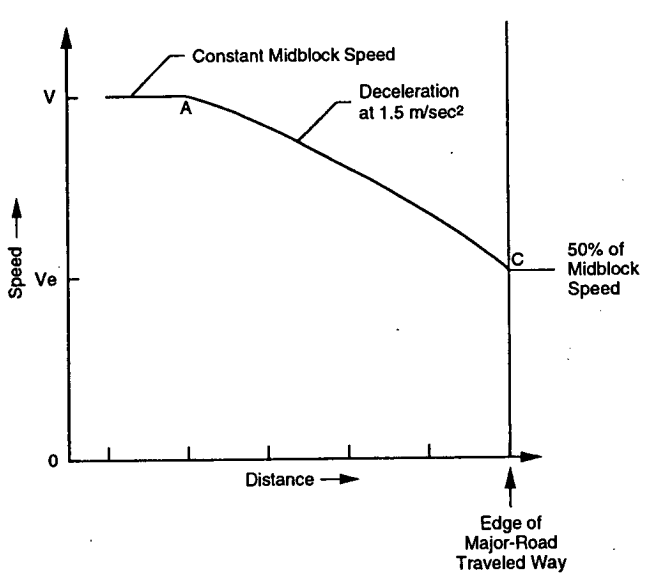


Figure 6. Typical driver-speed profile on the approach to an uncontrolled intersection.

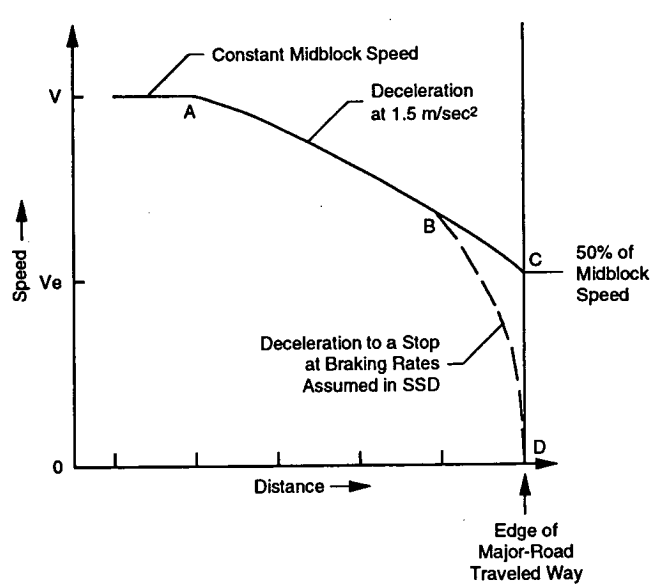


Figure 7. Typical driver-speed profile on the approach to an uncontrolled intersection showing deceleration to a stop if a potentially conflicting vehicle comes into view.

rate (1.5 m/sec^2 or 5 ft/sec^2, at most), and will begin soon enough for the vehicle to slow to 50 percent of its midblock running speed before reaching the intersection.

- Sufficient sight distance would be provided to allow each approaching driver 2.5 sec of perception-reaction time in which to detect potentially conflicting vehicles plus enough distance to stop before reaching the intersection if a potentially conflicting vehicle is present.
- The recommended sight distance is determined using the normal assumptions of the AASHTO SSD model, except that, for all or part of the perception-reaction time, the vehicle would be decelerating at the rate of 1.5 m/sec^2 (5.0 ft/sec^2) and the vehicle's braking to a stop would then begin at a speed lower than the midblock running speed at the braking rates assumed in the SSD model.

The assumption that vehicles slow to 50 percent of their midblock speed has been applied over the full range of design speeds considered. This behavior was observed at uncontrolled intersections on lower-speed urban and suburban residential streets, but no data are available concerning whether this same behavior occurs on higher-speed roads.

The following equations implement these assumptions. As the vehicle decelerates gradually, its speed at any point on the intersection approach is given by the equation

$$V_x^2 = V_e^2 - 2a_i X \tag{3}$$

where:

V_x = speed (m/sec or ft/sec) at any distance X from the intersection
V_e = speed (m/sec or ft/sec) at which vehicle would enter the intersection after decelerating (assumed: $V_e = 0.5$ V)
a_i = deceleration rate (m/sec^2 or ft/sec^2) on intersection approach before braking to a stop is initiated (assumed: $a_i = -1.5$ m/sec^2 or -5.0 ft/sec^2)
V = midblock running speed (m/sec or ft/sec)

Equation (3) represents line segment AC in Figure 7.

After the vehicle begins to brake to a stop, its speed at any point on the intersection approach is given by the equation:

$$V_x^2 = -2a_b X = 2fgX \tag{4}$$

where:

a_b = deceleration rate (m/sec^2 or ft/sec^2) during braking to a stop = fg
f = braking friction coefficient from AASHTO SSD model
g = acceleration of gravity (g = 9.8 m/sec^2 or 32.2 ft/sec^2)

Equation (4) represents line segment BD in Figure 7.

The distance from the intersection at which braking must begin for the vehicle to stop before reaching the intersection is given by

$$X_b = \frac{V_e^2}{2a_i + 2fg} \tag{5}$$

where:

X_b = distance from intersection at which braking begins (m or ft)

Equation (5) is derived simply by combining Equations (3) and (4) based on the assumption that braking begins where the lines represented by Equations (3) and (4) intersect (i.e., at Point B in Figure 7).

Once the value of X_b is known, the vehicle speed at which braking begins can be computed as

$$V_b = \sqrt{V_e^2 - 2a_i X_b} \tag{6}$$

where:

V_b = speed (m/sec or ft/sec) at which braking begins

Equation (6) is based directly on Equation (3).

Equations (5) and (6) must be solved iteratively because the value of the braking coefficient (f) in the current AASHTO SSD model varies as a function of speed. Thus, one must first solve Equation (5) for X_b using trial values of V_b and f. Then, Equation (6) is solved for V_b. If the computed value of V_b differs from the assumed value, then a new value of f is determined (based on the com-puted value of V_b) and Equations (5) and (6) are applied again. Agreement usually can be reached after only one or two iterations if a starting value of V_b is assumed to be equal to 75 percent of the midblock running speed (V).

Once the braking distance has been determined, the distance traveled during perception-reaction time can be computed. Two cases must be considered. As shown in Figure 8, one case occurs when the perception-reaction time occurs entirely during the initial deceleration [Equation (7)], while the other case occurs when perception-reaction time begins while the vehicle is still traveling at its midblock running speed [Equation (8)].

These two cases are represented by the following equations:

$$X_{pr} = V_b t_{pr} - 0.5 a_i t_{pr}^2, \text{ if } V_b - a_i t_{pr} \le V \tag{7}$$

$$X_{pr} = V t_{pr} - \frac{0.5(V_b - V)^2}{a_i}, \text{ if } V_b - a_i t_{pr} > V \tag{8}$$

where:

X_{pr} = distance traveled during perception-reaction time (m or ft)
t_{pr} = perception-reaction time (sec) (assumed: $t_{pr} = 2.5$ sec)

The required sight distance for an uncontrolled intersection approach would then be computed as:

$$ISD = X_{pr} + X_b \tag{9}$$

The ISD model for uncontrolled intersections represented by Equations (5) through (9) can be reformulated for speeds (V, V_b, and V_e) specified in miles per hour, as follows:

$$X_b = \frac{2.15V_e^2}{2a_i + 2fg} \tag{10}$$

$$V_b = \frac{\sqrt{2.15V_e^2 - 2a_iX_b}}{1.47} \tag{11}$$

$$X_{pr} = 1.47V_bt_{pr} - 0.5a_it_{pr}^2,\ if\ V_b - \frac{a_it_{pr}}{1.47} \le V \tag{12}$$

$$X_{pr} = 1.47Vt_{pr} - \frac{0.5(1.47V_b - 1.47V)^2}{a_i},\ if\ V_b - \frac{a_it_{pr}}{1.47} > V \tag{13}$$

$$ISD = X_{pr} + X_b \tag{14}$$

Similarly, the ISD model can be reformulated for speeds (V, V_b, and V_e) specified in kilometers per hour, as follows:

$$X_b = \frac{0.0772V_e^2}{2a_i + 2fg} \tag{15}$$

$$V_b = 3.6\sqrt{0.0772V_e^2 - 2a_iX_b} \tag{16}$$

$$X_{pr} = 0.278V_bt_{pr} - 0.5a_it_{pr}^2,\ if\ V_b - \frac{a_it_{pr}}{3.6} \le V \tag{17}$$

$$X_{pr} = 0.278Vt_{pr} - \frac{0.5(0.278V_b - 0.278V_m)^2}{a_i},$$

$$if\ V_b - \frac{a_it_{pr}}{3.6} > V \tag{18}$$

$$ISD = X_{pr} + X_b \tag{19}$$

Table 10 presents candidate ISD criteria for uncontrolled intersections based on these models in English and metric units, respectively. The tables show that the resulting ISD values are roughly comparable to current AASHTO policy for design speeds of 64 km/h (40 mph) or less. At higher speeds, the model indicates that more sight distance than required by current policy may be needed.

It was decided to recommend an ISD policy based on stopping rather than on adjusting speed because a model based on stopping is more conservative with respect to safety than a model based on adjusting speed. However, since field data show that some adjustment of speed occurs on approaches to uncontrolled intersections, this concept was incorporated in the stopping model. The rationale for the recommended model is explained more fully in the discussion of recommendations in the next section.

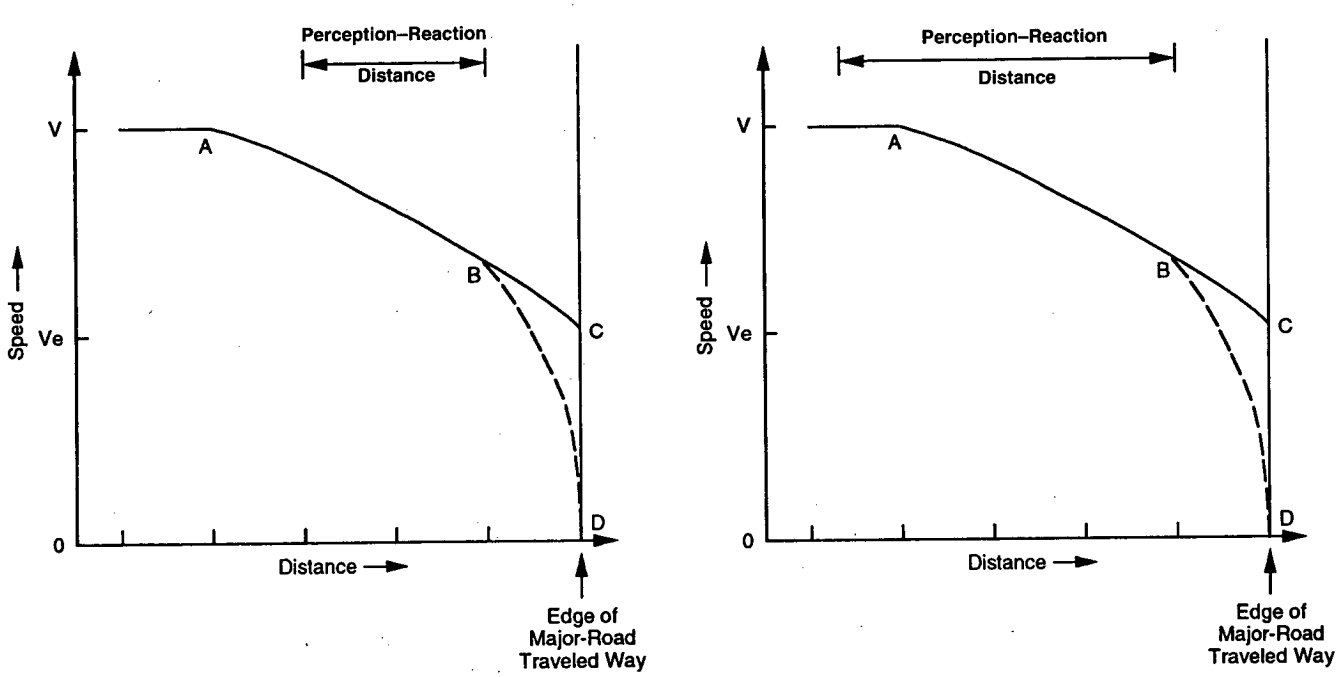


Figure 8. Two cases considered in computing perception-reaction distance for uncontrolled intersections.

TABLE 10 ISD Criteria for Uncontrolled Intersections Based on Stopping from a Reduced Speed

Design speed (km/h)	Sight distance (m)[a] Based on stopping from a reduced speed[b]	Sight distance (m)[a] Based on current AASHTO policy	Design speed (mph)	Sight distance (ft)[a] Based on stopping from a reduced speed[c]	Sight distance (ft)[a] Based on current AASHTO policy
20	20	20	10	45	45
30	25	25	15	60	70
40	30	35	20	80	90
50	40	40	25	95	110
60	50	50	30	120	130
70	65	60	35	140	155
80	80	65	40	170	180
90	95	75	50	255	220
100	120	85	60	350	260
110	140	90	70	480	310
120	165	100			

[a] Recommended length of the leg of the clear sight triangle along each intersecting roadway.
[b] Computed with Equations (15) through (19).
[c] Computed with Equations (10) through (14).

RECOMMENDATIONS

This section summarizes the recommended sight-distance policy for uncontrolled intersections and the rationale for that policy. A draft of an intersection sight distance policy for potential incorporation in a future edition of the AASHTO Green Book is presented in Appendix J.

Recommended Policy for Sight Distance to the Intersection

The driver should have a view of the intersection from a distance sufficient to stop, if necessary, before reaching the intersection. This normally is assured by the provision of SSD along each of the intersecting roadways. However, where sight distance of this length cannot be provided or where the presence of the intersection is not apparent, the installation of an advance warning sign should be considered.

Recommended Sight Distance Policy for Approach Sight Triangles at Uncontrolled Intersections

Clear sight triangles like those shown in Figure 9 should be provided in each quadrant of each approach to an uncontrolled intersection. The leg of the sight triangle along each approach (a or b) should be selected from the appropriate sight distance value in Table 10 for the design speed of that approach. If the design speed of either approach roadway is not known, it can be estimated by the 85th percentile mid-block running speed. Figure 10 compares the recommended

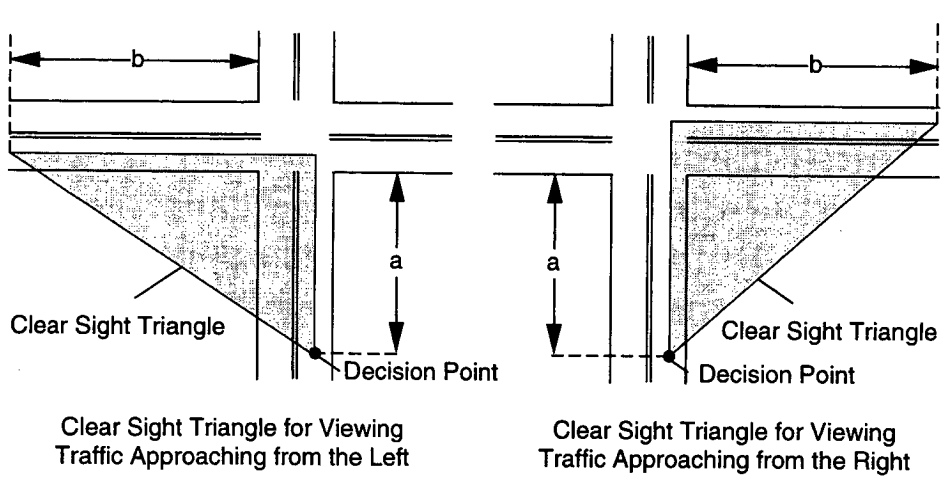


Figure 9. Approach sight triangles for uncontrolled intersections.

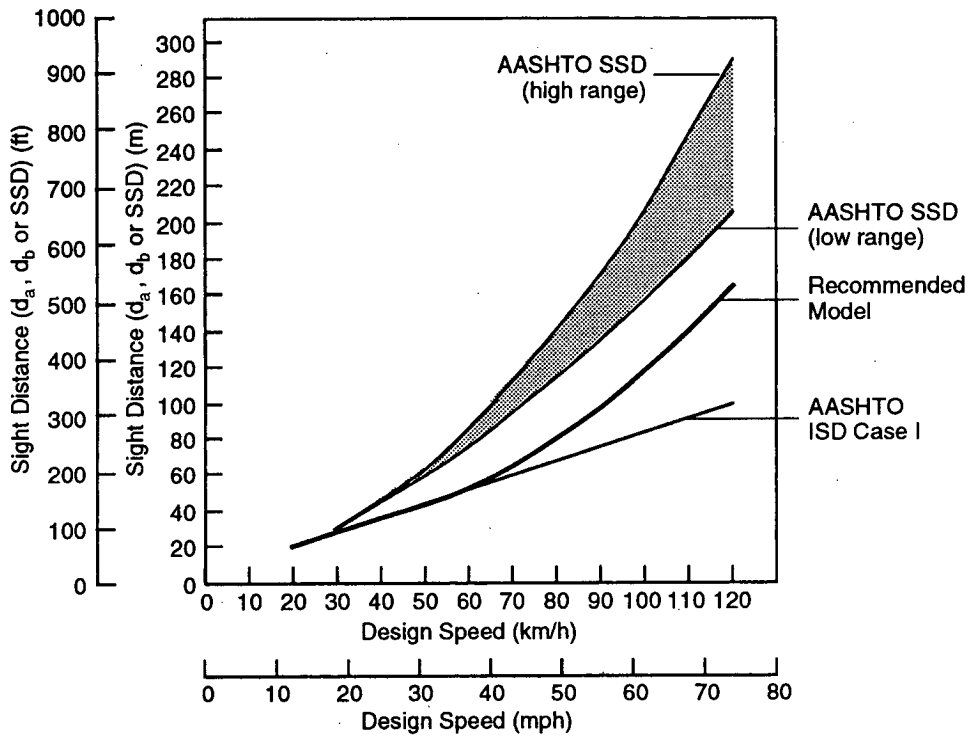


Figure 10. Recommended sight distances for uncontrolled intersections in comparison to current AASHTO policy.

sight-distance values to current AASHTO policy for ISD Case I and for SSD.

The vertex of the sight triangle on each approach represents a decision point at which the approaching driver must begin to consider the decision as to whether to stop before reaching the intersection.

No special sight-distance provisions for trucks are recommended because it is highly unlikely that there would be substantial truck volumes at an uncontrolled intersection since traffic volumes for all vehicle types at such intersections are generally low.

Identification of Sight Obstructions

The assessment of whether objects located within the clear sight triangles are sight obstructions should be based on a driver eye height of 1,080 mm (3.54 ft) and an object height of 1,080 mm (3.54 ft), as explained in Chapter 6. Any object found to be a sight obstruction should be removed or lowered, if possible.

Discussion of Recommendations

The recommended sight-distance model for uncontrolled intersections is based on stopping, rather than adjusting speed. A policy based on adjusting speed without stopping depends on the drivers of both vehicles to take the correct action; one driver must continue at constant speed or accelerate, while the other must slow down. A model based on adjusting speed without stopping cannot ensure that a collision can be avoided if the drivers of both potentially conflicting vehicles choose to slow down or stop.

The recommended model provides sufficient sight distance for either *or both* of the approaching vehicles to stop before reaching the intersection. The recommended model uses applicable concepts—perception-reaction time and braking coefficients—from the AASHTO SSD model. However, the recommended model, while based on stopping, incorporates the concept (supported by field observations) that drivers on approaches to uncontrolled intersections typically slow to 50 percent of the midblock running speed before reaching the intersection, whether a potentially conflicting vehicle comes into view or not. Thus, the recommended model combines the concepts of stopping and adjusting speed into a single model that is consistent with both the SSD model and with field observations of driver behavior at uncontrolled intersections.

For design speeds below 70 km/h (44 mph), the recommended sight-distance values are less than or equal to the current AASHTO values for ISD Case I. At a design speed of 70 km/h (44 mph), the recommended sight-distance

value exceeds current policy by only 5 m (16 ft). Thus, the recommendations presented here should require very little change in current policies at lower-speed intersections, including most urban and suburban locations.

The recommended model would require highway agencies to consider longer sight distances than current policy at higher-speed uncontrolled intersections, particularly rural locations. At a typical rural speed of 90 km/h (56 mph), the recommended model requires 26 percent more sight distance than current AASHTO policy. At 120 km/h (75 mph), the highest speed addressed in the Green Book, the recommended model would require 65 percent more sight distance than current policy; however, uncontrolled intersections are extremely rare on highways operating at such high speeds. Nevertheless, if an uncontrolled intersection were operated at such a speed, the current AASHTO model does not appear appropriate because it does not provide sufficient sight distance for the situation in which both approach drivers slow down or brake to a stop. Furthermore, the Poisson arrival model shows that a situation in which two vehicles arrive on intersecting approaches within 2 sec of one another can be expected to occur 113 times per year where the intersecting roads have volumes of 100 veh/day and 1,800 times per year where the intersecting roads have volumes of 400 veh/day.

The field studies which established that drivers normally slow down on approaches to uncontrolled intersections did not include higher-speed rural intersections, so it is not known whether the typical driver speed profile shown in Figure 6 applies to such intersections. Although the field studies did not establish driver speed profiles at higher-speed rural intersections, the recommended model would result in increased sight distances at such intersections which should contribute to their safe operation.

While the policy recommended for inclusion does entail longer sight distances than current policy for uncontrolled intersections on higher-speed roadways, it should be recognized that the Green Book is applicable only to newly constructed intersections. The Green Book does not require that existing locations be improved. The policies of most highway agencies concerning resurfacing, restoration, and rehabilitation projects could require an improvement in ISD at an existing intersection if there were a documented accident pattern associated with the existing design.

Sight-distance policies for Stop- and Yield-controlled intersections make allowance for the sight distance required by vehicles that have stopped to enter or cross the major road. No similar allowance has been made for uncontrolled intersections because the Poisson arrival model indicates that, at low-volume intersections, if a vehicle has slowed or stopped to accommodate one vehicle on an intersecting approach, the appearance of a second vehicle on that same approach would be an extremely rare event.

Incorporation of Changes to the SSD Model Recommended by Fambro et al.

Fambro et al. has recommended several changes to AASHTO SSD policy in "Determination of Stopping Sight Distances," in a forthcoming NCHRP Project. The recommendations for ISD policy for uncontrolled intersections

TABLE 11 ISD Criteria for Uncontrolled Intersections Based on Stopping from a Reduced Speed (modified for consistency with the stopping sight distance model recommended by Fambro et al.)

Design speed (km/h)	Sight distance (m)[a,b]	Design speed (mph)	Sight distance (ft)[a,c]
20	20	10	45
30	25	15	65
40	35	20	85
50	45	25	105
60	55	30	130
70	65	35	155
80	75	40	185
90	90	50	250
100	105	60	320
110	120	70	400
120	135		

[a] Recommended length of the leg of the clear sight triangle along each intersecting roadway. This table is for use to replace the recommended values in Table 10 if the stopping sight distance recommendations of Fambro et al. are adopted as AASHTO policy.

[b] Computed with Equations (15) through (19) assuming a deceleration rate (fg) equal to 3.4 m/sec^2.

[c] Computed with Equations (10) through (14), assuming a deceleration rate (fg) equal to 11.1 ft/sec^2.

presented above are based, in part, on parameter values used in the current AASHTO SSD policy. If the recommendations made by Fambro et al. are adopted by AASHTO for use in SSD policy, corresponding changes to the ISD recommendations presented above for uncontrolled intersections should be considered for consistency.

The stopping sight distance model recommended by Fambro et al. is:

$$SSD = 0.278Vt_{pr} + \frac{0.039V^2}{a_b} \tag{20}$$

where:

SSD = stopping sight distance (m)
V = initial speed (km/h)
t_{pr} = perception-reaction time (sec)
a_b = deceleration rate during braking to a stop (m/sec^2)

The parameter values recommended by Fambro et al. for use in Equation (20) are 2.5 sec for perception-reaction time and 3.4 m/sec^2 (11.1 ft/sec^2) for deceleration rate. Thus, the perception-reaction time recommended for use in the revised SSD model is the same as used in the current AASHTO SSD model and the same as used in the recommended ISD model for uncontrolled intersections.

The only substantive change from the current AASHTO SSD model is that Equation (20) incorporates a deceleration rate for controlled braking rather than a locked-wheel braking coefficient. The deceleration rate recommended by Fambro et al. is 3.4 m/sec^2 (11.1 ft/sec^2) for all initial speeds. Thus, unlike the braking friction coefficient in the current AASHTO SSD model, the recommended deceleration rate does not vary with speed. The deceleration rate in Equation (20) is equivalent to the product fg in Equations (4), (5), (10), and (15).

Fambro et al. recommends that the initial speed should be equal to the roadway's anticipated 85th percentile operating speed. We would make a similar, but slightly modified, recommendation that the initial speed should be equal to the design speed of the roadway and that the roadway design speed should be based on the roadway's anticipated operating conditions. This formulation avoids the use of the term "operating speed" which may have different meanings to different readers.

Table 11 presents revised ISD criteria for uncontrolled intersections that should be considered as a replacement for the criteria in Table 10 if the stopping sight distance recommendations of Fambro et al. are adopted as AASHTO policy. The differences between Tables 10 and 11 are minor with slightly longer sight distances at low speeds and moderately shorter sight distances at higher speeds.

CHAPTER 3

EVALUATION OF ISD POLICY FOR INTERSECTIONS WITH STOP CONTROL ON THE MINOR ROAD

This chapter presents an evaluation of ISD policy for intersections with Stop control on the minor road.

CURRENT AASHTO POLICY

Current AASHTO geometric design policy for sight distance at intersections with Stop control on the minor road consists of three cases:

- Case IIIA—Stop control on minor road: crossing maneuver;
- Case IIIB—Stop control on minor road: left-turn maneuver; and
- Case IIIC—Stop control on minor road: right-turn maneuver.

In contrast to ISD Case I, discussed in the previous chapter, which involved clear sight triangles for vehicles approaching an intersection, ISD Case III involves clear sight triangles for stopped vehicles departing from the intersection. Each of these three cases is discussed below:

ISD Case IIIA—Stop Control on Minor Road: Crossing Maneuver

ISD Case IIIA addresses the sight distance required for a vehicle to accelerate and cross the major road from a stopped position on the minor-road approach to a Stop-controlled intersection.

The AASHTO Green Book states that the sight distance for a crossing maneuver is based on the time it takes for the stopped vehicle to clear the intersection and the distance that a vehicle will travel along the major road at its design speed in that amount of time. However, the time element also includes a perception-reaction and vehicle transmission actuation component. The sight distance for Case IIIA was calculated in the 1984 and 1990 Green Books from the equation:

$$ISD = 1.47V(J + t_a) \quad (21)$$

where:

ISD = sight distance to the left (d_1) or sight distance to the right (d_2) along the major road from the intersection required for the minor-road vehicle to cross the major road (ft)

V = design speed of the major road (mph)

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

t_a = time required to accelerate and traverse the distance S to clear the major-road pavement (sec) (determined from the distance vs. time relationships for passenger cars, single-unit trucks, and WB-15 trucks)

S = D + W + L, the distance that the crossing vehicle must travel to clear the major road (ft)

D = distances from the near edge of pavement to the front of a stopped vehicle (ft) (assumed: D = 10 ft)

W = intersection width along the path of the crossing vehicle (ft)

L = overall length of minor-road vehicle (ft)

Figure 11 illustrates these distances and vehicle positions for Case IIIA.

Equation (22) presents the metric equivalent to Equation (21), based on the metric AASHTO policy that appears in the 1994 edition of the AASHTO Green Book (*1*):

$$ISD = 0.278V(J + t_a) \quad (22)$$

where the speed (V) is specified in kilometers per hour and the sight distance (ISD) is determined in meters.

ISD is measured from a driver eye height of 1,070 mm (3.50 ft) to an object height of 1,300 mm (4.25 ft) above the pavement, intended to represent the height of a potentially conflicting vehicle.

The J term comprises the time allowed for scanning in both directions by the vehicle operator to determine if there is a sufficient gap in the major-road traffic to initiate and complete the crossing maneuver safely and the time required to shift gears or actuate an automatic transmission. For the J term, a value of 2.0 sec is assumed. This value is assumed to be a constant, since there are no data concerning whether it might be appropriate to vary J from the assumed value of 2.0 sec for changing conditions (e.g., vehicle type, operator type, and so on).

Values for the term t_a are based on acceleration performance data for both passenger cars and trucks. The acceleration data for passenger cars were updated in the 1990 Green Book based on data in *NCHRP Report 270* (*10*). The values

of t_a can be read directly from Figure 12 (which reproduces Figure IX-33 from the AASHTO Green Book) for a given distance S for nearly level conditions.

As stated above, S is the distance that the crossing vehicle must travel to clear the major highway. It comprises the variables D, W, and L. A value of 3 m (10 ft) is assumed for the distance (D) from the near edge of pavement to the front of a stopped vehicle. Values for the width (W) of the pavement that must be crossed depend on the number of lanes on the major roadway. Lane widths of 3.6 m (12 ft) are used. A calculation example for ISD Case IIIA is provided in the 1994 Green Book (Figure IX-37) to illustrate the treatment of median widths in the selection of appropriate values for W. Green Book values for overall vehicle length (L) are 5.8, 9.1, 15.2, and 16.7 m (19, 30, 50, and 55 ft) for the P, SU, WB-12, and WB-15 design vehicles, respectively.

It should be noted that the longest truck explicitly considered in the determination of S for this case is the WB-15 truck with an overall length of 16.7 m (55 ft). Substantially longer trucks are common in today's vehicle fleet.

When design speeds on the major roadway exceed 80 km/h (50 mph), the SSD requirements for some intersection widths (W) will be greater than the corresponding ISD for passenger cars. When this occurs, provision of SSD along the major road usually ensures that the required ISD for Case IIIA will be available. ISD for Case IIIA and SSD are measured in different ways. SSD is measured along the major road. ISD is measured from the driver's eye location in the normal stopping position for a minor-road vehicle. The front of the minor-road vehicle is assumed to be 3 m (10 ft) from the edge of the major-road traveled way, and the driver's eye location is assumed to be 3 m (10 ft) from the front of the vehicle. Thus, ISD Case IIIA is measured from a position 6 m (20 ft) from the edge of the major-road traveled way. Since SSD is based on a 150-mm (6-in) object and ISD is based on a 1,300-mm (4.25-ft) object, Case IIIA ISD will usually (but not necessarily always) be available when SSD is available.

ISD Case IIIB—Stop Control on Minor Road: Left-Turn Maneuver

ISD Case IIIB involves a situation in which a vehicle is stopped on the minor road awaiting an opportunity to complete a left-turn maneuver by clearing traffic approaching from the left and then enters the traffic stream approaching from the right. Figure 13 illustrates this situation.

AASHTO policy states that a vehicle accelerating from a stop to turn left into a major highway should have, as a minimum, sufficient sight distance so that a collision will not occur if a vehicle approaching from the right and traveling at the design speed of the major road appears when the turning vehicle begins its maneuver. The turning vehicle should be able to accelerate to a safe running speed by the time the approaching vehicle closes to within a specified tailgate distance or minimum separation.

The 1984 Green Book (*3*) presented criteria for ISD Case IIIB based on two alternative scenarios for the behavior of the major-road vehicle. Under the first scenario, the major-road vehicle continues to travel at the design speed of the major roadway (Curve B-2a); and the turning vehicle must accelerate to that speed; under the second scenario, the major-road vehicle reduces speed from the design speed to the average running speed of the major road (Curve B-2b) and the minor-road vehicle, therefore, has to accelerate only to the average

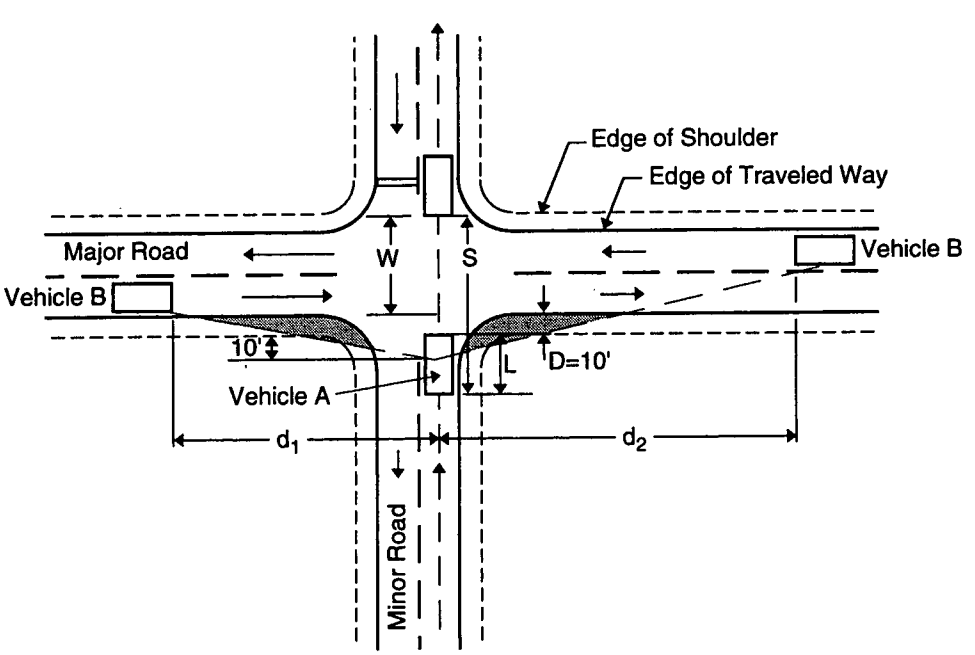


Figure 11. Minimum clear sight triangles used for ISD Case IIIA.

running speed. In the 1990 Green Book (*2*), the first scenario was eliminated and the second scenario was changed so that the major-road vehicle reduces speed from the design speed of the major road to 85 percent of the design speed. The scenario based on deceleration by the major-road vehicle to 85 percent of the design speed has been retained in the 1994 Green Book (*1*).

Figure 14 compares several key curves for ISD Case IIIB. The figure illustrates:

- The range of values specified in the AASHTO SSD policy;
- Curve B-1, which represents the sight distance to the left required for Vehicle A to cross the near lane of traffic on a two-lane highway;
- Curve B-$^1/_4$-lane+median, which represents the sight distance to the left required for Vehicle A to cross both near lanes of a traffic on a four-lane highway;

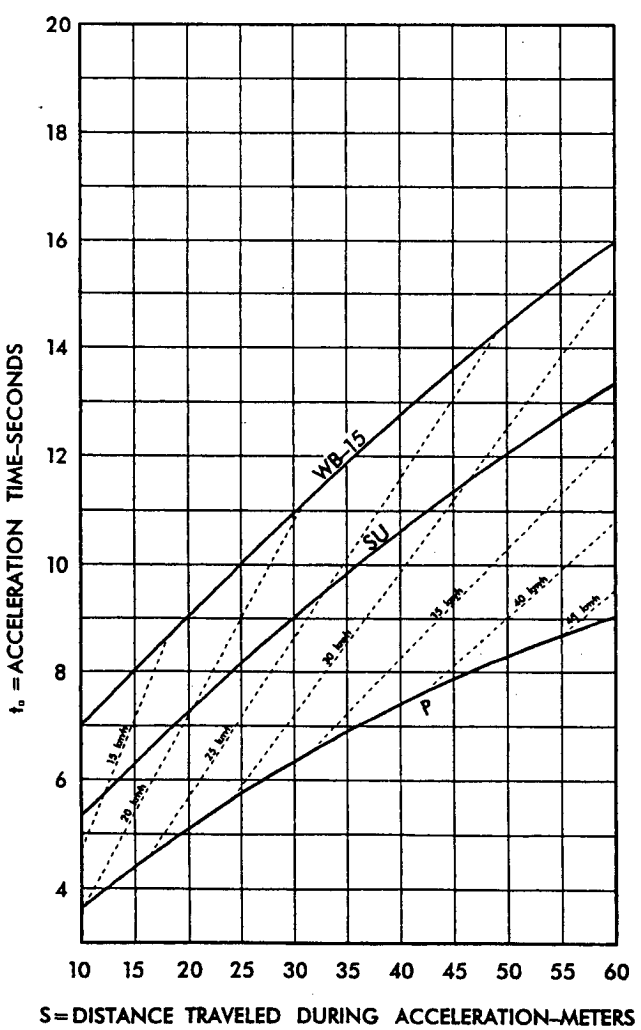


Figure 12. Time required for passenger cars, single-unit trucks, and WB-15 trucks to accelerate a given distance (1).

- Curve B-2a & Ca (1984), which represents the sight distance to the right required by Vehicle A to accelerate to the design speed of the major road and complete its left turn without being overtaken by Vehicle B, which travels at a constant speed equal to the design speed of the major road;
- Curve B-2b & Cb (1984), which represents the sight distance to the right required by Vehicle A to accelerate to the average running speed of the major road and to complete its left turn without being overtaken by Vehicle B, which reduces speed from the design speed of the major road to the average running speed; and
- Curve B-2b & Cb (1990 and 1994), which represents the sight distance to the right required by Vehicle A to accelerate to 85 percent of the design speed of the major road and to complete its left turn without being overtaken by Vehicle B, which reduces speed from the design speed of the major road to 85 percent of the design speed.

The B-1 curves represent the required sight distance to the left and the B-2a and B-2b curves represent the sight distance to the right required to complete a left turn. The B-2a curve has been dropped from the 1990 and 1994 Green Books and only the B-1 curve and the version of the B-2b curve based on deceleration to 85 percent of the design speed have been retained.

The sight distance to the left required by the B-1 curve is seldom critical because it is always less than the sight distance to the left required to complete a right turn (see subsequent discussion of ISD Case IIIC). At low speeds, sight distance required by the B-1 curve exceeds the SSD required along the major road. The sight distance represented by the B-1 curve will become the critical sight distance only in a quadrant on an intersection approach where right turns are prohibited.

The sight distances to the right required by the B-2b curve in the 1990 and 1994 Green Book are 4 percent to 23 percent less than those in the 1984 Green Book.

The 1984 Green Book did not adequately explain how Curves B-2a and B-2b were derived. Harwood et al. (*9*) have developed equations that can reproduce the 1984 Green Book curves within 8 percent accuracy.

Curve B-2b in the 1990 Green Book can be reproduced with the following sequence of equations:

$$Q = (1.47)(0.95)V(t_a + J) \tag{23}$$

$$h = P - \left(\frac{\pi R}{2} - R\right) - (1.47)(0.85)Vt_{vg} - L_a \tag{24}$$

$$ISD = Q - h \tag{25}$$

where:

ISD = sight distance (ft) along the major road from the intersection required for Vehicle A to depart from a stop, accelerate to 85 percent of the major-road design speed, and complete a turn to the left without being

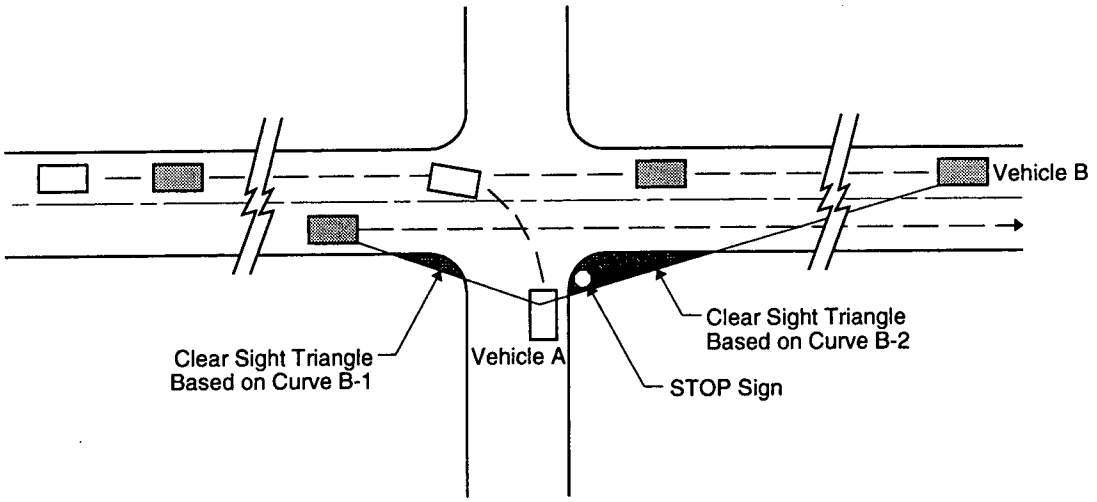


Figure 13. Minimum clear sight triangles used for ISD Case IIIB.

overtaken by Vehicle B approaching from the right traveling at the design speed and decelerating to a speed equal to 85 percent of the major-road design speed

Q = distance traveled by Vehicle B (ft)

h = distance along the major road traveled by Vehicle B from the midpoint of the turning lane on the minor road to the end of maneuver (ft)

V = design speed of the major road (mph)

t_a = acceleration time of Vehicle A (sec) (Note: the value of t_a is determined from Table IX-7 in the 1990 Green Book based on a speed equal to 85 percent of the major-road design speed)

J = perception-reaction time of driver of Vehicle A (sec) (assumed: J = 2.0 sec)

P = acceleration distance of Vehicle A (ft) (Note: the value of P is determined from Table IX-7 in the 1990 Green Book based on a speed equal to 85 percent of the

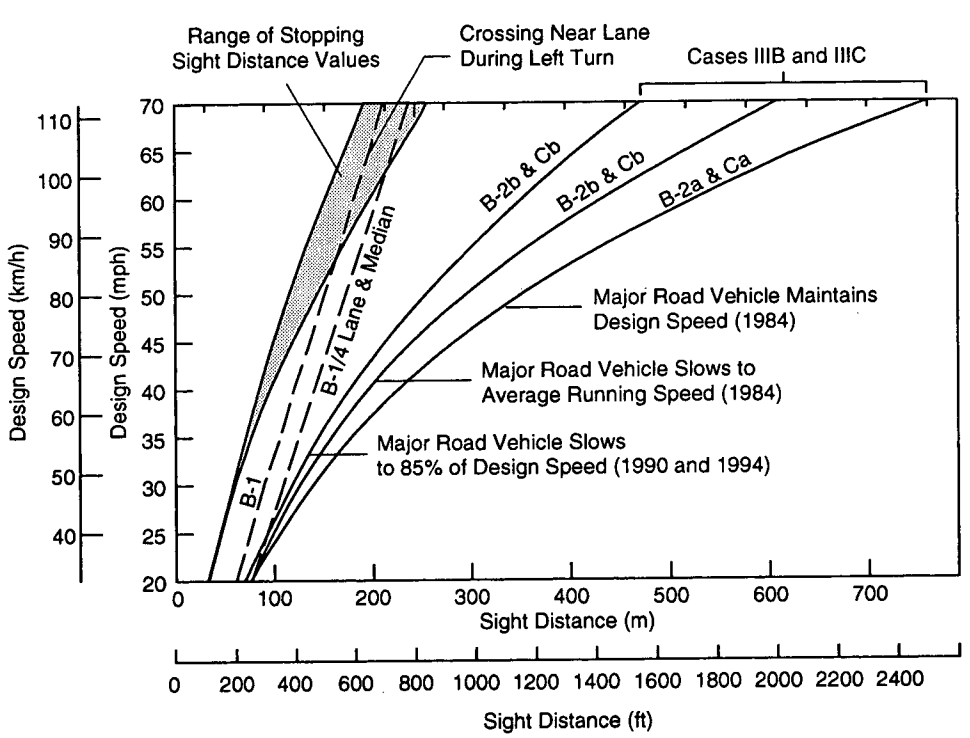


Figure 14. Sight distance curves for ISD Cases IIIB and IIIC from 1984, 1990, and 1994 AASHTO Green Books (1,2,3).

major-road design speed)

R = radius of turn for Vehicle A (ft) (assumed: R = 28 ft for left turns)

t_{vg} = vehicle gap time (sec) (assumed: t_{vg} = 2.0 sec)

L_a = length of Vehicle A (ft) (Note: assumed L_a = 19 ft for a passenger car)

Equation (23) incorporates the assumption that the average speed of Vehicle B during its deceleration maneuver is 95 percent of the major-road design speed. This assumption is consistent with the initial speed of Vehicle B equal to the major-road design speed and its final speed equal to 85 percent of the major-road design speed.

The $\pi R/2 - R$ term is a correction that is made because Vehicle A travels further along its curved turning path than it would if all its travel were parallel to the centerline of the major road. Figure 15 illustrates the rationale for this term. The assumed turning radius (R) of 8.5 m (28 ft) is determined as the sum of three distances: a 3.1-m (10-ft) setback from the edge of the major-road traveled way to the front of the stopped vehicle; the 3.6-m (12-ft) width of the near lane of the major-road traveled way; and one-half the width of far lane of the major-road traveled way (1.8 m or 6 ft). Thus, Curve B-2b is clearly based on the assumption that the major road is a two-lane, two-way highway.

The 1990 Green Book implied that t_{vg} has a value of 1.9 sec. However, a value of t_{vg} equal to 2.0 sec must be used to reproduce the B-2b curve in the Green Book.

Sight distance for Case IIIB is measured from an assumed stopping position with the front of Vehicle A located 3 m (10 ft) from the edge of the major road. The driver's eye in Vehicle A is assumed to be 3 m (10 ft) behind the front of the vehicle. Thus, the driver's eye is assumed to be 6 m (20 ft) from the edge of the minor road. As in all ISD cases, the height of the driver's eye is assumed to be 1,070 mm (3.50 ft) and the height of the vehicle to be seen is assumed to be 1,300 mm (4.25 ft).

Curve B-2b in the 1994 Green book can be reproduced with the following sequence of equations:

$$Q = (0.278)(0.95)V(t_a + J) \tag{26}$$

$$h = P - D - 1.5w_\ell - \frac{\pi R}{2} + 2R - (0.278)(0.85)Vt_{vg} - L_a \tag{27}$$

$$ISD = Q - h \tag{28}$$

where:

ISD = sight distance (m) along the major road from the intersection required for Vehicle A to depart from a stop, accelerate to 85 percent of the major-road design speed, and complete a turn to the left without being overtaken by Vehicle B approaching from the right traveling at the design speed and decelerating to a speed equal to 85 percent of the major-road design speed

Q = distance traveled by Vehicle B (m)

h = distance along the major road traveled by Vehicle B from the midpoint of the turning lane on the minor road to the end of the maneuver (m)

V = design speed of the major road (km/h)

t_a = acceleration time of Vehicle A (sec) (Note: The value of t_a is determined from Table IX-8 in the 1994 Green Book based on a speed equal to 85 percent of the major-road design speed)

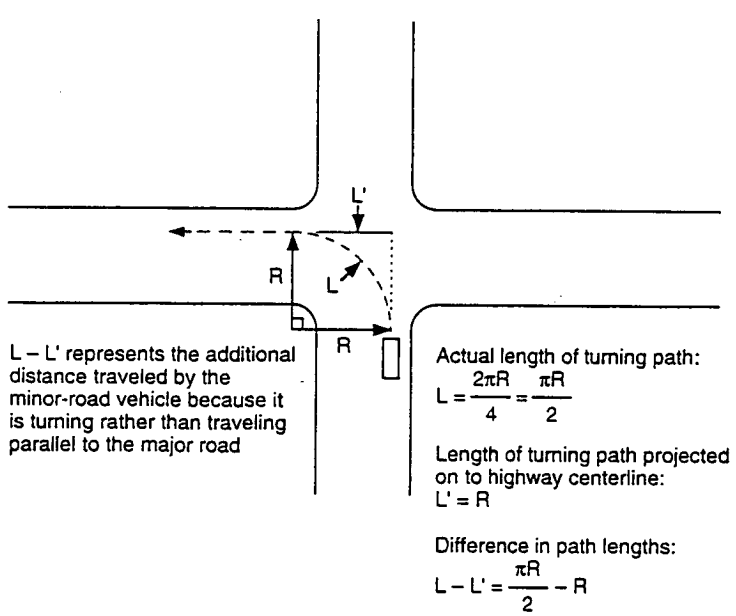


Figure 15. Derivation of $\pi R/2 - R$ term in Equation (24).

J = perception-reaction time of driver of Vehicle A (sec) (assumed: J = 2.0 sec)
P = acceleration distance of Vehicle A (m) (Note: The value of P is determined from Table IX-8 in the 1994 Green Book based on a speed equal to 85 percent of the major-road design speed)
D = distance from front of Vehicle A when stopped on the minor-road approach to the edge of the major-road traveled way (m) (assumed: D = 3 m)
w_l = lane width on major road (m) (assumed: w_l = 3.6 m)
R = radius of turn for Vehicle A (m) (assumed: R = 8.5 m for left turns)
t_{vg} = vehicle time gap (sec) (assumed: t_{vg} = 2.0 sec)
L_a = length of Vehicle A (m) (assumed: L_a = 5.8 m for a passenger car)

Figure 16, which is based on Green Book Figure IX-39, illustrates the terms in Equations (26) through (28).

Equation (27) is conceptually equivalent to Equation (24) because of the implicit assumption in the 1990 Green Book that the turning radius for a left turn onto a two-lane highway was given by:

$$R = D + 1.5w_\ell \tag{29}$$

The incorporation of explicit terms for D and w_l provides greater flexibility in dealing with roadways of different width. For example, the 1994 Green Book provides an example in which Equation (27) is modified, as follows, for a left turn onto a four-lane divided highway:

$$h = P - D - 2w_\ell - w_m - \frac{\pi R}{2} + 2R - (0.278)(0.85)Vt_{vg} - L_a \tag{30}$$

where:

w_m = median width on divided highway (m)

It seems to the authors that, for consistency with the concept developed in Equation (27), the coefficient of w_l in Equation (30) should be 2.5 rather than 2.

ISD Case IIIC—Stop Control on Minor Road: Right-Turn Maneuver

ISD Case IIIC involves a situation in which a vehicle is stopped on the minor road waiting for an opportunity turn right onto the major road. This situation is illustrated in Figure 17. AASHTO criteria state that a vehicle accelerating from a stop to turn right onto a major highway should have, as a minimum, sufficient sight distance so that a collision will not occur if a vehicle approaching from the left and traveling at the design speed of the major road appears when the turning vehicle begins its maneuver. The turning vehicle should be able to accelerate to a safe running speed by the time the approaching vehicle closes to within a specified distance behind the turning vehicle.

The current AASHTO criteria for Case IIIC are identical to the criteria for Case IIIB, except that Curve B-1 is not relevant to right turns. In the 1984 Green Book (*3*), the same two alternative scenarios were considered as for Case IIIB: the major-road vehicle was assumed to continue at the design speed or to decelerate to the average running speed. In the 1990 and 1994 Green Books (*1,2*), the major-road vehicle in Case IIIC, as in Case IIIB, is assumed to decelerate from the design speed of the major road to 85 percent of the design speed.

The AASHTO model for ISD Case IIIC in the 1994 Green Book differs from the model for Case IIIB in only two respects. First, for a right turn, the coefficient of w_l in Equation (27) is 0.5 rather than 1.5, because there is no lane to be crossed before turning. Second, the radius of turn is assumed to be 7.6 m (25 ft), rather than 8.5 m (28 ft). These differences are illustrated in the diagram in Figure 18, based on Green Book Figure IX-42.

The differences between the models for Cases IIIB and IIIC result in a sight-distance value that is 3.3 m (11 ft) less for right turns than for left turns. Because this difference is so small, AASHTO policy chooses to ignore it and applies the Case IIIB criteria to Case IIIC as well. This is also the reason that the curves in Figure 14 are labeled Curve B-2a & Ca and Curve B-2b & Cb, to call attention to the fact that they apply to both Cases IIIB and IIIC.

CURRENT HIGHWAY AGENCY POLICIES

Current state and local highway agency policies for each ISD case for Stop-controlled intersections are summarized below. International policies for ISD design are summarized in Appendix C.

Crossing Maneuver

Table 12 summarizes the design policies used by state and local highway agencies for the crossing maneuver at a Stop-controlled intersection. The table shows that 85 percent of state highway agencies use either the 1984 or 1990 Green Book policies for Case IIIA. Furthermore, the 1984 and 1990 Green Book policies are identical except for updated acceleration rates for passenger cars in the 1990 policy. It should be noted that the survey on which Table 12 was based (see Appendix B) was conducted before publication of the 1994 Green Book.

Only three state highway agencies have design policies for Case IIIA that differ from AASHTO. These policies are as follows:

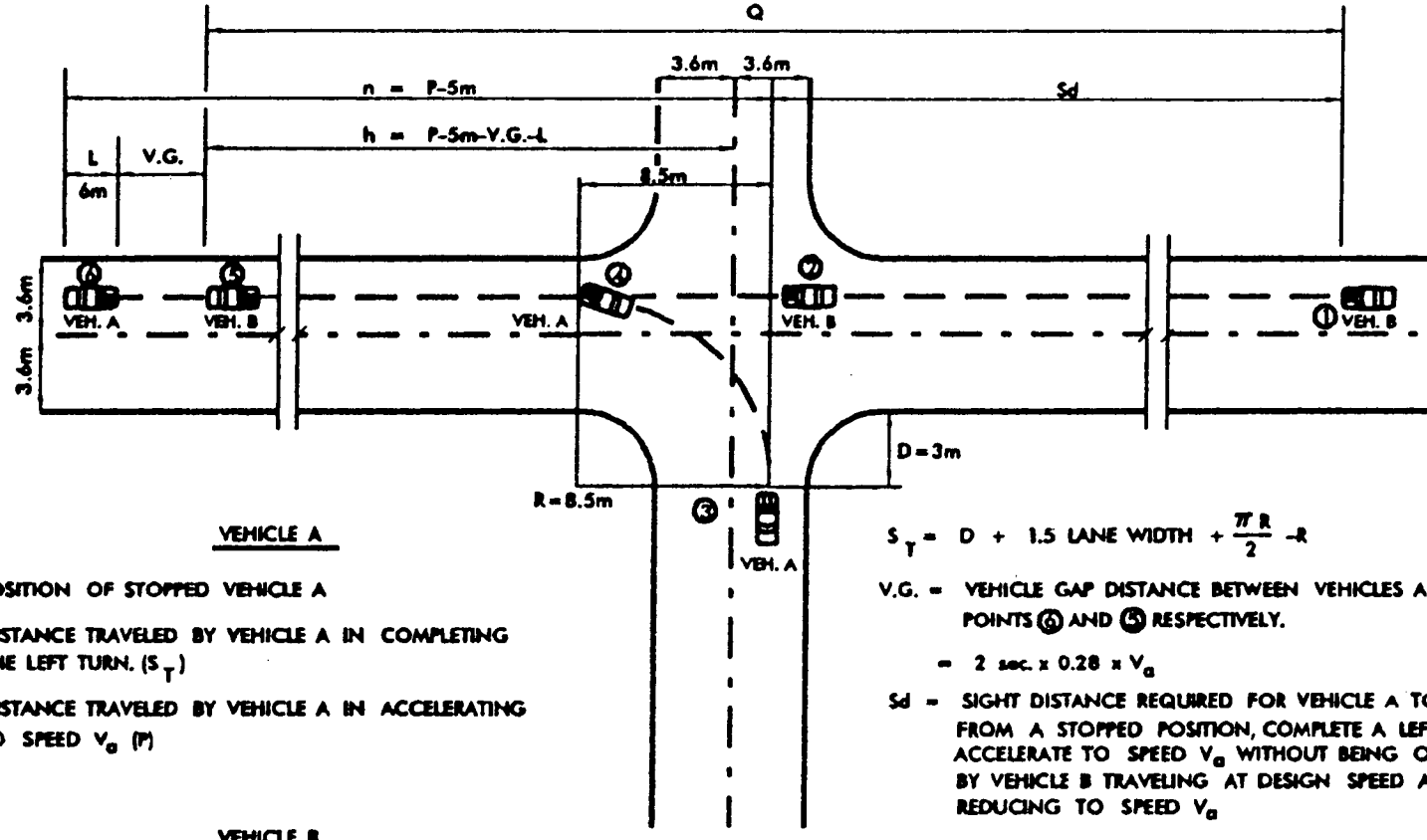
- One state has developed a policy for Case IIIA based on acceleration rates for the minor-road vehicle that differ from AASHTO.

VELOCITY OF VEH. A at ③ = 0 VELOCITY OF VEH. B at ① = D.S.

VELOCITY OF VEH. A at ⑥ = 0 VELOCITY OF VEH. B at ⑤ = Va

VEHICLE A AND B ARE 6M IN LENGTH

LEVEL CONDITIONS



VEHICLE A

③ POSITION OF STOPPED VEHICLE A

③-④ DISTANCE TRAVELED BY VEHICLE A IN COMPLETING THE LEFT TURN. (S_T)

③-⑥ DISTANCE TRAVELED BY VEHICLE A IN ACCELERATING TO SPEED V_a (P)

VEHICLE B

① POSITION OF VEHICLE B TRAVELING AT DESIGN SPEED 2 SECONDS BEFORE VEHICLE A STARTS HIS DEPARTURE MOVEMENT.

①-⑤ DISTANCE TRAVELED BY VEHICLE B WHILE REDUCING TO SPEED V_a AND BY NOT ENCROACHING CLOSER THAN V.G. TO VEHICLE A WHEN VEHICLE A HAS REACHED POINT ⑥ (Q)

S_T = D + 1.5 LANE WIDTH + $\frac{\pi R}{2}$ -R

V.G. = VEHICLE GAP DISTANCE BETWEEN VEHICLES A AND B AT POINTS ⑥ AND ⑤ RESPECTIVELY.

= 2 sec. x 0.28 x V_a

Sd = SIGHT DISTANCE REQUIRED FOR VEHICLE A TO DEPART FROM A STOPPED POSITION, COMPLETE A LEFT TURN AND ACCELERATE TO SPEED V_a WITHOUT BEING OVERTAKEN BY VEHICLE B TRAVELING AT DESIGN SPEED AND REDUCING TO SPEED V_a

V_a = 85% DESIGN SPEED (km/h)

n = P-S_T +R

h = P-S_T +R-V.G.-L

SD = Q-h

Figure 16. Illustration of AASHTO model for Case IIIB (stopped vehicle turning left onto two-lane major highway) (1).

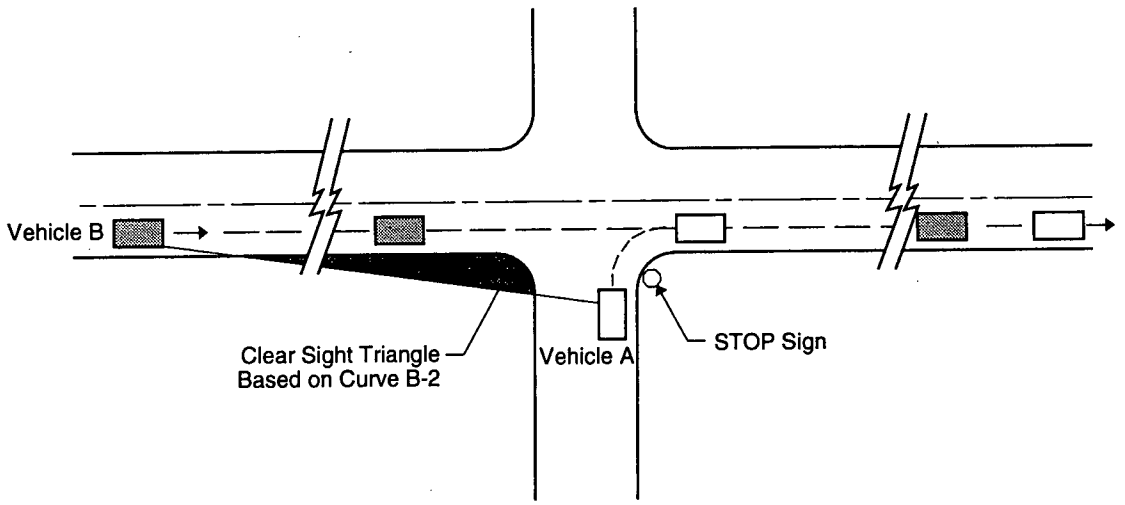


Figure 17. Minimum clear sight triangle used for ISD Case IIIC.

- One state uses a sight-distance policy that they refer to as alignment criteria for all aspects of Case III. These criteria provide sight distances that are longer than the AASHTO criteria for Case IIIA, but shorter than the AASHTO criteria for Cases IIIB and IIIC.
- One state uses a policy that is presented in Table 5 for all intersections and all sight-distance cases.

Four state highway agencies do not consider ISD Case IIIA. Three of these four agencies have stated explicitly that they do not consider Case IIIA because Cases IIIB and IIIC are more critical (i.e., they require greater ISD).

Approximately 57 percent of local agencies in urban areas and 35 percent of local agencies in rural areas use either the 1984 or 1990 AASHTO policy for ISD Case IIIA. Five local agencies (one urban and four rural) report that they follow the policy of their state highway agency that differs from AASHTO. Approximately 24 percent of local agencies in urban areas and 13 percent of local agencies in rural areas report that they have adopted their own local policies for Case IIIA. These policies general consist of either:

- Provision of SSD along the major roadway, or
- Provision of a clear sight triangle of specified dimensions by means of a local ordinance that restricts sight obstructions on private property.

Approximately 16 percent of local agencies in urban areas and 35 percent of local agencies in rural areas do not consider Case IIIA. In urban areas, this may be because Cases IIIB and IIIC are considered to be more critical. In rural areas, this is more likely to represent the lack of any policy for ISD Case III. The percentage of rural agencies that do not consider Cases IIIB and IIIC is higher than the percentage of agencies that do not consider Case IIIA.

Left-Turn Maneuver

Based on the survey of state highway agencies presented in Appendix B, nearly 75 percent of state highway agencies use the AASHTO policy for ISD Case IIIB. Approximately 50 percent of the states use the policy in the 1990 Green Book and 25 percent of the states use the 1984 Green Book. It should be noted that this survey was conducted prior to publication of the 1994 Green Book.

Table 13 summarizes the design policies used by state and local highway agencies for ISD Case IIIB. Table 14 compares the ISD design values used by various highway agencies; the table is presented in English units because most of the state highway agency policies on which it is based were published in English units. A total of 12 states (25 percent) have adopted their own policy for Case IIIB that differs from AASHTO. These modified policies are described below:

- Three states use the AASHTO policy for Case IIIA instead of the AASHTO policy for Case IIIB.
- Two states have developed new equations for ISD Case IIIB that differ from the AASHTO model.
- One state uses a gap-acceptance procedure based on provision of sight distance for a 7.5-sec gap.
- One state also uses minimum ISD requirements for Case IIIB based on a gap-acceptance procedure with a minimum gap of 8 sec for passenger cars and 12 sec for trucks. This state also uses desirable ISD values based on the equations developed by Fitzpatrick et al. (*11*) to approximate the 1990 Green Book curves for Case IIIB. This state has, however, modified these equations for trucks. Rather than the speed reduction to 85 percent of the design speed as specified in the Green Book, this state uses a speed reduction to 65 percent of the design speed when a truck is considered as the turning vehicle.

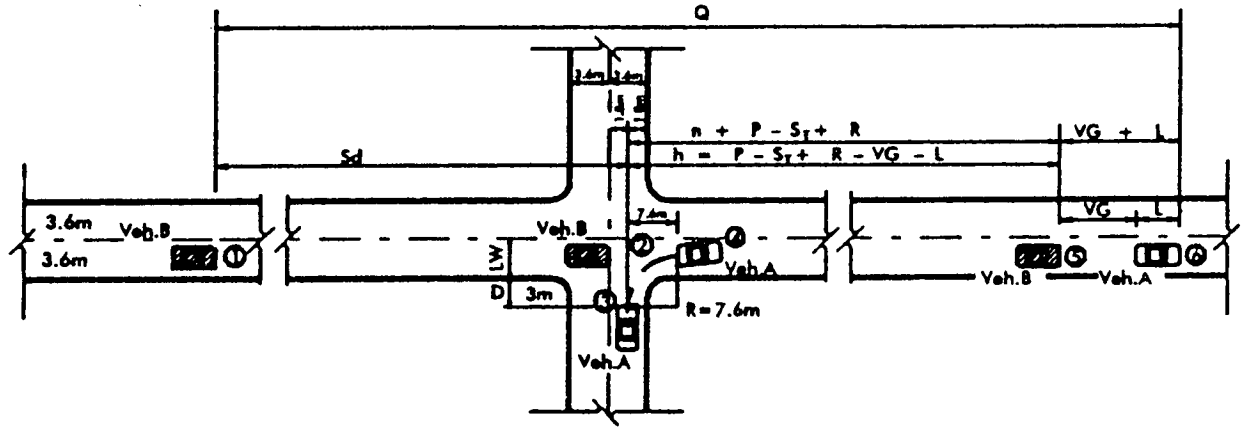
VELOCITY OF VEH. A AT ③ = 0

VELOCITY OF VEH. A AT ⑥ = Va

VELOCITY OF VEH. B AT ① = D.S.

VELOCITY OF VEH. B AT ⑤ = Va

VEHICLE A AND B ARE 6m IN LENGTH
LEVEL CONDITIONS



S_r = D + 1/2 L.W. + π R/2 - R

Sd = SIGHT DISTANCE FOR VEHICLE A TO DEPART FROM STOP POSITION, COMPLETE A RIGHT TURN AND ACCELERATE WITHOUT BEING OVERTAKEN BY VEHICLE B TRAVELING AT DESIGN SPEED AND REDUCING TO SPEED Va.

VG = VEHICLE GAP DISTANCE BETWEEN VEHICLES A AND B AT POINTS ⑥ AND ⑤ RESPECTIVELY.

= SPEED OF VEHICLE B AT POINT ⑤ (km/h) x 2 sec. x 0.28

h = P - S_r + R - VG - L

n = P - S_r + R

Sd = Q - h

Va = 85% DESIGN SPEED (km/h).

VEHICLE A

③ POSITION OF STOPPED VEHICLE A

③-④ DISTANCE TRAVELED BY VEHICLE A IN COMPLETING THE RIGHT HAND TURN. (Sd)

③-⑥ DISTANCE TRAVELED BY VEHICLE A FROM STOP POSITION AND ACCELERATING TO SPEED Va. (P)

VEHICLE B

① POSITION OF VEHICLE B TRAVELING AT DESIGN SPEED 2 SECONDS BEFORE VEHICLE A STARTS THE DEPARTURE MOVEMENT.

①-⑤ DISTANCE TRAVELED BY VEHICLE B WHILE REDUCING TO SPEED Va AND BY NOT ENCHROACHING CLOSER THAN VEHICLE GAP DISTANCE TO VEHICLE A WHEN VEHICLE A HAS REACHED POINT ⑥. (Q)

Figure 18. Illustration of AASHTO model for Case IIIC (stopped vehicle turning right into a major highway) (1).

TABLE 12 Summary of Highway Agency Design Policies for ISD Case 111A

Design policy	Number (percentage) of agencies					
	State highway agencies		Local agencies			
			Urban		Rural	
1990 AASHTO Green Book	27	(57.4)	15	(40.5)	6	(26.1)
1984 AASHTO Green Book	13	(27.7)	6	(16.2)	2	(8.7)
Own state policy	3	(6.4)	1	(2.7)	4[a]	(17.4)
Own local policy	–	–	9	(24.3)	2	(13.0)
Don't consider this case	4	(8.5)	6	(16.2)	8	(34.8)
	47		37		23	

[a] These local agencies use design policies for ISD Case IIIA based on the policy of the state highway agencies in their states.

- One state uses a sight distance model similar to the equations developed by Fitzpatrick et al. (*11*), but allows the designer to select the appropriate speed reduction for the major-road vehicle at particular locations. Sight-distance criteria are tabulated for deceleration by the major-road vehicle to speeds in the range from 40 percent to 100 percent of the major-road design speed. A speed reduction equal to 50 percent of the major-road design speed is recommended for design.
- One state uses a sight-distance policy that it refers to as alignment criteria for all aspects of Case III. These sight distances are longer than those for AASHTO Case IIIA, but shorter than those for AASHTO Case IIIB.
- One state uses the sight-distance values presented in Table 5 for *all* intersections, including Stop-controlled intersections.
- One state uses a modification of Case IIIB, which includes sight-distance criteria for trucks.

TABLE 13 Summary of Highway Agency Design Policies for ISD Case IIIB

Design policy	Number (percentage) of agencies					
	State highway agencies		Local agencies			
			Urban		Rural	
1990 AASHTO Green Book	23	(48.9)	15	(40.5)	5	(21.7)
1984 AASHTO Green Book	12	(25.5)	6	(16.2)	1	(4.3)
Own state policy	12	(25.5)	1[a]	(2.7)	4[a]	(17.4)
Own local policy	–	–	11	(29.7)	3	(13.0)
Don't consider this case	0	(0.0)	4	(10.8)	10	(43.5)
	47		37		23	

[a] These local agencies use design policies for ISD Case IIIB based on the policy of the state highway agencies in their states.

- One state has no formal policy for Case IIIB, but uses the AASHTO Case IIIB as desirable sight-distance values to be attained, if possible.

Approximately 57 percent of local agencies in urban areas and 26 percent of local agencies in rural areas report that they use the AASHTO policy for Case IIIB. Five local agencies reported that they follow the Case IIIB policy of their state highway agency which differs from AASHTO.

Approximately 30 percent of local agencies in urban areas and 13 percent of local agencies in rural areas have adopted their own policies for Case III. Typical policies include:

- Use of Case IIIA rather than Case IIIB;
- Use of Curve B-1 in Figure 14 rather than Curve B-2b & Cb;
- Use of specific gap acceptance values; and
- Use of a local ordinance that requires property owners to keep their property clear of sight obstructions within a specified minimum sight triangle.

Eleven percent of local agencies in urban areas and 44 percent of local agencies in rural areas do not have a policy for ISD Case IIIB.

Right-Turn Maneuver

Table 15 summarizes the current policies of state and local agencies for ISD Case IIIC. Most highway agencies recognize the similarity of Cases IIIB and IIIC and use the same criteria for both. The only exception is one state highway agency that uses the 1984 Green Book policy for Case IIIB, but claims not to consider Case IIIC.

ASSESSMENT OF CURRENT POLICIES

A thorough assessment of the current AASHTO and highway agency policies for ISD design at intersections with Stop control on the minor road was conducted. This assessment included a sensitivity analysis of the current AASHTO models and an evaluation of the degree of support from previous research for the functional form of each model and for the values assumed for particular parameters in those models. This assessment led to the development of alternative ISD models which are discussed in the next section.

A general finding of this assessment is that the Green Book fails to make clear that—unlike SSD criteria, which are clearly based on perceived safety requirements—ISD criteria are intended more to ensure desirable operations than to provide minimum values that are necessary for safety. For example, a Stop-controlled intersection could be operated safely if the drivers of vehicles on a major-road approach and drivers of vehicles at the stop line on the minor road can see one another from a distance equivalent to the appropriate SSD criteria. If a minor-road vehicle

TABLE 14 Comparison of AASHTO Criteria for Case IIIB with State Highway Agency Criteria That Differ from AASHTO

Agency	Required intersection sight distance (ft)										
	Design speed (mph)										
	20	25	30	35	40	45	50	55	60	65	70
AASHTO B–1 (1990)	200	260	310	360	410	460	510	560	620	670	720
AASHTO B–2b & Cb (1990)	220	290	370	470	570	700	830	980	1150	1340	1560
AASHTO B–2a & Ca (1984)	250	340	450	580	750	950	1180	1440	1750	2100	2500
AASHTO B–2b & Cb (1984)	250	330	410	520	660	840	1030	1240	1470	1720	2000
AASHTO SSD–High Range	125	150	200	250	325	400	475	550	650	725	850
AASHTO SSD–Low Range	125	150	200	225	275	325	400	450	525	550	625
Alaska (desirable ISD)	300	370	450	580	750	950	1180	1450	1750	2100	--
Alaska (minimum ISD)	125	150	200	225	275	325	400	450	525	550	--
California (desirable)	220	275	330	385	440	495	550	605	660	715	770
California (minimum)	125	150	200	250	300	360	430	500	580	660	750
Indiana (proposed)	198	265	344	433	533	646	778	910	1075	--	--
Maine	--	300	380	480	580	710	840	990	1150	1350	1550
Michigan	125	150	200	250	325	400	475	550	650	725	850
Mississippi (2–lane)	--	--	420	490	560	620	680	740	800	860	920
Mississippi (4–lane)	--	--	480	550	620	700	780	860	940	1000	1060
Mississippi (6–lane)	--	--	540	620	700	780	860	940	1020	1110	1200
Missouri	--	--	160	160	185	200	220	240	260	285	310
Ohio	300	375	450	500	575	625	700	750	825	875	950
Oklahoma (desirable/PCs)	220	280	355	440	525	635	765	895	1035	1190	1375
Oklahoma (minimum/PCs)	220	280	355	415	470	530	590	645	705	765	825
Oklahoma (desirable/trucks)	325	400	495	595	705	845	995	1185	1420	--	--
Oklahoma (minimum/trucks)	325	400	495	595	705	795	880	970	1060	1145	1235
South Carolina	205	270	320	390	455	530	615	700	800	895	1000
Virginia (2–lane major road)	200	250	300	350	400	450	500	550	600	650	700
Virginia (4–lane major road)	240	300	350	425	475	525	600	650	700	750	825
Washington (P or SU veh)	--	390	460	540	680	855	1030	1255	1480	1740	2000
Washington (WB–50 veh)	--	510	620	720	820	925	1030	1255	1480	1740	2000
Washington (WB–63 veh)	--	590	710	830	940	1060	1180	1330	1480	1740	2000
Range of state criteria											
Highest	325	590	710	830	940	1060	1180	1450	1750	2100	2000
Lowest	125	150	160	160	185	200	220	240	260	285	310

TABLE 15 Summary of Highway Agency Design Policies for ISD Case IIIC

Design policy	Number (percentage) of agencies					
	State highway agencies		Local agencies			
			Urban		Rural	
1990 AASHTO Green Book	22	(46.8)	16	(43.2)	5	(21.7)
1984 AASHTO Green Book	12	(25.5)	5	(13.5)	1	(4.3)
Own state policy	12	(25.5)	1	(2.7)	4[a]	(17.4)
Own local policy	–	–	10	(27.0)	3	(13.0)
Don't consider this case	1	(2.1)	5	(13.5)	10	(43.5)
	47		37		23	

[a] These local agencies use design policies for ISD Case IIIC based on the policy of the state highway agencies in their states.

pulled into the path of a major-road vehicle, the driver of the major-road vehicle would have sufficient time to come to a stop, even under adverse conditions, as long as the driver of the major-road vehicle first sees the minor-road vehicle begin its maneuver from a distance at least equal to SSD. However, while safe, this minimal type of design would be undesirable from an operational viewpoint, because the major-road vehicle might need to stop to accommodate the minor-road vehicle, which is exactly the reverse of the intended operations for the intersection. It would be appropriate for the Green Book to state explicitly that ISD criteria for Stop-controlled intersections are longer than SSD, not because the ISD values are critical to safety, but to ensure that the intersection operates smoothly, in accordance with the designer's intentions, with minor-road vehicles waiting until they can see that they can proceed safely without forcing a major-road vehicle to stop.

Another general finding of the assessment is that the existing AASHTO ISD criteria for Stop-controlled intersections give too little attention to ISD requirements of trucks. The AASHTO criteria for the crossing maneuver (ISD Case IIIA) consider truck requirements explicitly, but the criteria for left- and right-turn maneuvers (ISD Cases IIIB and IIIC) do not. It would seem more appropriate for the Green Book to present criteria for consideration in the design of Stop-controlled intersections with a substantial number of trucks either crossing or entering the major road.

The assessment of each of the ISD cases for Stop-controlled intersections is presented below.

Crossing Maneuver

The current AASHTO policy appears to place too much emphasis on the ISD criteria for the crossing maneuver. Since the ISD values for the left- and right-turn maneuver exceed those for the crossing maneuver, any intersection that is designed to provide sufficient sight distance for left- and right-turn maneuvers should also provide sufficient sight distance for the crossing maneuver. As stated above, three state highway agencies have recognized this by excluding ISD Case IIIA from their own design policies on the grounds that the sight-distance requirements for ISD Cases IIIB and IIIC are greater. Case IIIA may have been retained in the Green Book primarily for historical reasons because, until 1984, Case IIIA was the only ISD policy for Stop-controlled intersections, rather than because the crossing maneuver is frequently a critical design criterion.

The research team has been able to identify only three (rather unusual) geometric situations where Case IIIA could be more critical than Cases IIIB and IIIC. The first situation, shown in Figure 19, has steep upgrades on both the minor-road approach on which the stopped vehicle is waiting and also on the departing roadway on the opposite minor-road leg. Here, if the grade is sufficiently steep, trucks or other vehicles with higher weight-to-power ratios may need t_a values that are greater than those shown in Figure 12. In an extreme case, this could yield ISD values for Case IIIA greater than the sight distances for Case IIIB or IIIC maneuvers. The second, also highly unlikely, situation in which ISD Case IIIA could control the design would be at an intersection where the crossing maneuver is the only legal maneuver. This could occur at the intersection of a two-way or one-way minor road with two one-way major roadway legs flowing into the intersection where the through roadways (rather than the one-way roadways) are controlled by Stop signs (Figure 20). This situation could also occur if the left-turn maneuver onto the major road were prohibited by signing. In this instance, the minor-road vehicle could not turn left or right, but could only proceed through the intersection onto the other minor-road leg. It is not clear that any such intersections actually exist, but if one did, Case IIIA would be the controlling ISD design criterion. A third situation in which the Case IIIA sight distance could become greater than those for Cases IIIB and IIIC would occur if, at a particular intersection, there were substantial truck volumes making crossing maneuvers, but very few trucks making left or right turns from the minor road.

Based on this assessment, it appears more appropriate to make the crossing maneuver a special consideration in design of Stop-controlled intersections to be checked in specific situations like those described above rather than to retain it as a separate ISD case.

While it appears appropriate to diminish the importance of the crossing maneuver in ISD policy, the model currently used to determine the sight-distance requirements for the crossing maneuver [see Equations (21) and (22)] appears to remain very appropriate.

The AASHTO Green Book policy describes the J term in Equations (21) and (22) as representing a two-stage process involving searching for an acceptable gap in the major-road traffic stream and then actuating the clutch or "actuating an automatic shift." The Green Book recognizes that the times for these two stages may overlap, but suggests that they be treated serially so that the resulting time estimate is conserv-

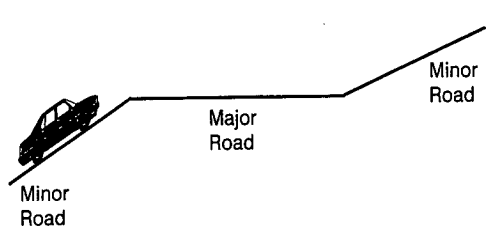


Figure 19. Steep upgrades on minor road can make ISD Case IIIA more critical.

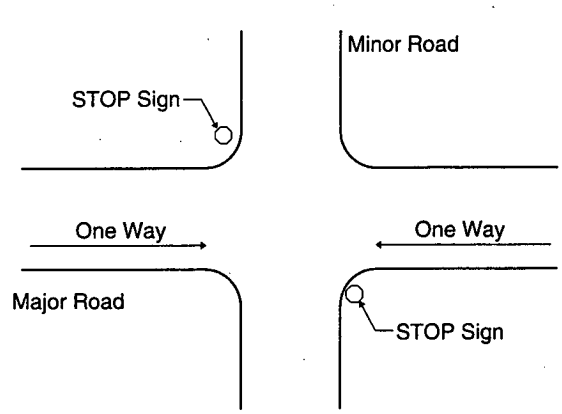


Figure 20. ISD Case IIIA is critical at an intersection where the crossing maneuver is the only maneuver legally permitted.

ative. The language about actuating a clutch is left over from earlier AASHTO policies when most passenger cars had manual transmissions; today this language is typically applicable only to trucks and a very few passenger cars. Actuation of an automatic transmission usually means only the transfer of the driver's foot from the brake pedal to the accelerator. It might be more clear to define the J term simply as perception-reaction time and state that the required reaction could be either actuation of the clutch or transfer of the driver's foot to the accelerator.

Research by McGee and Hooper (*7*), Hostetter et al. (*12*), and, most recently, by Lerner et al. (*13*) confirms the choice of 2.0 sec as the value of perception-reaction time for the crossing maneuver in Equations (21) and (22). Furthermore, a sensitivity analysis indicates that Equations (21) and (22) are much more sensitive to design speed and acceleration time than to perception-reaction time. Therefore, modification of the 2.0-sec perception-reaction time criterion would have relatively little effect on the resulting ISD criteria.

Left-Turn Maneuver

Figure 14 indicates that Case IIIB can require a sight distance of over 450 m (1,500 ft) for passenger cars at higher design speeds. The Green Book states that the required sight distances for trucks making left turns are substantially longer than for passenger cars. However, there has been no attempt in the Green Book to quantify these sight-distance requirements for trucks. An analysis of truck ISD requirements for Case IIIB was performed by Harwood et al. (*9*) using field data collected in a recent FHWA study and equations that approximately reproduce the sight-distance values in the 1984 Green Book. This analysis found that over 900 m (3,000 ft) of sight distance could be required for trucks at higher design speeds. Most intersections do not have available sight distances that even approach 900 m (3,000 ft), yet they operate safely every day for both trucks and passenger cars. These findings suggest that the assumptions in the AASHTO model for Case IIIB are too conservative or that the model itself may need to be revised.

A major concern with the current AASHTO model for Case IIIB is that it is based on an assumption concerning the deceleration behavior of the major-road vehicle that is not based on field data. However, even casual observation of intersections suggests that major-road vehicles often slow by more than 15 percent when a minor-road vehicle enters the highway in front of them.

The current model for AASHTO Case IIIB assumes a perception-reaction time of 2.0 sec for the minor-road driver. This value is supported for both younger and older drivers in recent research by Lerner et al. (*13*).

A shortcoming of the current AASHTO model is that it does not include explicit consideration of the perception-reaction time required by the major-road driver before beginning to decelerate after a vehicle turning onto the major road begins to move. The perception-reaction time requirements of the major-road driver have not been considered previously except in an ISD model developed by the Connecticut Department of Transportation. A conservative assumption for the perception-reaction time of the major-road driver would be 2.0 sec, the same value assumed for the minor-road driver. However, this is an alerted condition and the actual perception-reaction time requirements could be much shorter because the major-road driver may see the intersection, and may even see the minor-road vehicle at the stop line, before the minor-road vehicle begins to move.

The assumption of a vehicle gap time or minimum separation of 2.0 sec appears quite conservative because the minimum gap should exist only briefly if the turning vehicle continues to accelerate.

As in the crossing maneuver, there appears to be little justification for inclusion in AASHTO policy of the B-1 curve, which represents the sight distance required to cross the near lane of a highway when turning left. The sight distance required by the B-1 curve is always less than sight distance required to turn right. Thus, the B-1 curve is applicable only to situations in which the near lane must be crossed in turning left, but right turns into the near lane are not permitted. In addition, the B-1 curve has created some confusion because at least one local agency has used the B-1 curve as a general sight-distance policy rather than just for its intended limited purpose.

Right-Turn Maneuver

The assessment concluded that AASHTO's current policy, based on the premise that the sight-distance requirements for ISD Cases IIIB and IIIC differ so little that the same criteria can be applied to each, is very appropriate. However, this means that the shortcomings of the current AASHTO model for the left-turn maneuver discussed above are also applicable to the right-turn maneuver.

ALTERNATIVE ISD MODELS AND METHODOLOGIES

The following ISD models and methodologies were identified and considered in the early stages of the research as alternatives to the current AASHTO model of the ISD requirements for left- and right-turn maneuvers:

- Current AASHTO model using variable speed reduction;
- An alternative model developed by Fitzpatrick et al. (*11*);
- An alternative model developed by the Arizona Department of Transportation;
- An alternative model developed by the Connecticut Department of Transportation;
- An alternative model based on gap acceptance; and

- A modified AASHTO model with explicit terms for perception- reaction time and deceleration rate of major-road vehicle.

Each of these alternatives was evaluated during the research. The current AASHTO model with a variable speed reduction, the Fitzpatrick et al. model, the Arizona model, and the Connecticut model were each dropped from consideration, as explained in Appendix G, because the modified AASHTO model developed in this study appeared to be conceptually superior. The gap-acceptance model was also retained for further consideration. Each of these alternative approaches is explained more fully below.

Alternative Model Based on Gap Acceptance

Gap acceptance provides one alternative approach to ISD Case IIIB. The rationale for gap acceptance as an ISD criterion is that if drivers will accept a specific critical gap, such as 7.5 sec, in the major-road traffic stream when making a turning maneuver, and if such turning maneuvers are routinely completed safely, then sufficient ISD should be provided to enable drivers to identify that critical gap. Thus, field determination of safe critical gaps can lead directly to ISD criteria based on gap acceptance.

At least two state highway agencies currently use design criteria based on gap acceptance for ISD Case IIIB. The California Department of Transportation uses criteria for Case IIIB based on a 7.5-sec gap in major-road traffic. The Oklahoma Department of Transportation uses an 8.0-sec gap for turning maneuvers by passenger cars and a 12.0-sec gap for turning maneuvers by trucks in Case IIIB. In addition, France and Sweden both use ISD design policies based on gap acceptance.

A gap-acceptance model for ISD Case IIIB can be simply formulated as:

$$ISD = 1.47 * V * G \tag{31}$$

where:

ISD = sight distance (ft)
V = major-road design speed (mph)
G = specified critical gap (sec)

An equivalent metric expression for gap acceptance is

$$ISD = 0.278 * V * G \tag{32}$$

where the major-road design speed is specified in kilometers per hour and the resulting sight distance is in meters.

Table 16 shows the current AASHTO ISD criteria for Case IIIB interpreted as time gaps in the major-road traffic stream. The table shows that the gaps currently required by AASHTO policy range from 7.8 sec (roughly equivalent to the California and Oklahoma policies) at a 30-km/h (19-mph) design speed to 14.9 sec at a 110-km/h (69-mph) design speed. The rationale for gap acceptance as an ISD criterion is that drivers safely accept gaps much shorter than 14.9 sec routinely, even on higher-speed roadways.

Lerner et al. (*13*) performed a field study of gap acceptance evaluations by older and younger drivers for through, right, and left-turn maneuvers. This study was based on asking test subjects seated in a stationary vehicle to identify gaps that they would and would not accept; no actual turning or crossing maneuvers were made. The gap for right turns was based only on the presence of vehicles approaching from the left in the near lane. For through and left-turn movements, the gaps were based on the presence of vehicles approaching either from the left in the near lane, from the right in the far lane, or both. The results of this study indicate that the critical gap accepted by 50 percent of drivers across all age groups and driving conditions is approximately 7 sec. The 85th percentile gap was found to be approximately 11 sec. Neither the maneuver (left, right, through), the time of day (day, night), or the site was found

TABLE 16 Acceptable Gaps in Major-Road Traffic Implied by Current AASHTO Policy for ISD Case IIIB

Design speed (km/h)	Required ISD for Case IIIB (m)[a]	Corresponding time gap (sec)	Design speed (mph)	Required ISD for Case IIIB (ft)[b]	Corresponding time gap (sec)
30	65	7.8	20	220	7.5
40	90	8.1	25	290	7.9
50	120	8.6	30	370	8.4
60	160	9.6	35	470	9.2
70	205	10.5	40	570	9.7
80	255	11.5	45	700	10.6
90	310	12.4	50	830	11.3
100	380	13.7	55	980	12.2
110	455	14.9	60	1,150	13.1
			65	1,340	14.1
			70	1,560	15.2

[a] For a left turn onto a two-lane highway based on 1994 Green Book.
[b] For a left turn onto a two-lane highway based on 1990 Green Book.

TABLE 17 Critical Gaps Measured in the Field for Passenger Cars at Two-Way Stop-Controlled Intersections by Kyte et al.

Maneuver	Two-lane major road	Multilane major road
Right-turn from minor road	5.5	6.8
Left-turn from minor road	6.4	7.3
Crossing maneuver from minor road	6.4	7.6
Left-turn from major road	4.2	4.2

to affect significantly the gap selected. The sites included both 56-km/h (35-mph) and 80-km/h (50-mph) roadways. In the daytime, male drivers were observed to accept gaps that were shorter by approximately 1 sec than those accepted by females. Small age differences were apparent in the daytime, but not at night.

An extensive field study of gap-acceptance behavior was recently completed by Kyte et al. in NCHRP Project 3-46 ("Capacity Analysis of Unsignalized Intersections," unpublished). The objective of the Kyte study was to develop capacity and level-of-service procedures based on gap acceptance for inclusion in the chapter on unsignalized intersections in the *Highway Capacity Manual* (*HCM*) (*15*). Kyte et al. observed gap-acceptance behavior of drivers at 44 two-way Stop-controlled intersections in five different regions of the United States, including 30 three-leg intersections and 14 four-leg intersections. The gap-acceptance data were analyzed using the maximum likelihood method through a procedure developed by Troutbeck (*15*).

Table 17 summarizes the critical gaps measured in the field by Kyte et al. An analysis of variance was performed to develop adjustment factors for use in the *HCM* account for the effects of the number of lanes on the major road, the type of vehicle making the maneuver, the grade of the minor road, and the differences between four-leg and three-leg intersections. Table 18 summarizes the values of critical gaps and adjustment factors that are proposed for use in the unsignalized intersection procedures in the *HCM*.

The Lerner et al. (*13*) and Kyte et al. studies, which show critical gaps in the range from 6 to 7 sec, suggest that gap acceptance holds promise as a basis for ISD at Stop-controlled intersections. Field studies, reported in the next section, were conducted to help in further quantifying the appropriate critical gap for use in ISD criteria.

Modified AASHTO Model with Explicit Perception-Reaction Time and Deceleration Rate Terms for the Major-Road Vehicle

A modified AASHTO model of the sight-distance requirements for left turns at Stop-controlled intersections was developed based on the following assumptions for a scenario involving two passenger cars:

- Following perception-reaction time (J_a), the minor-road

TABLE 18 Recommended Critical Gap Values for Use In Unsignalized Intersection Capacity Analysis by Kyte el al.

Geometry	Maneuver			
	Right-turn from minor road	Left-turn from minor-road	Crossing maneuver from minor road	Left-turn from major road
Base critical gap value for passenger cars at single-lane sites	6.2 sec	7.1 sec	6.5 sec	4.1 sec
Multilane adjustment	0.7 sec	0.4 sec	0.5 sec	--
Heavy-vehicle adjustment[a]	1.0–2.0 sec			
Adjustment for minor-road grade	0.1 sec/%	0.2 sec/%	0.2 sec/%	--
Adjustment for three-leg site	--	--	--	-0.7 sec

[a] Heavy-vehicle adjustment is +1.0 sec for single-lane sites and +2.0 sec for multilane sites.

vehicle (Vehicle A) starts and accelerates through its turn in accordance with the acceleration times (t_a) in Table IX-8 in the 1994 Green Book.

- Following perception-reaction time (J_b) after Vehicle A begins to move, the major-road vehicle (Vehicle B) begins to decelerate from the major-road design speed at a constant maximum deceleration rate that can be specified as an input parameter to the model. Vehicle B continues to decelerate until it reaches the speed of Vehicle A or until it reaches a minimum acceptable speed (or maximum acceptable speed reduction) that can be specified as an input parameter to the model.
- The vehicle gap or minimum separation between Vehicles A and B at the end of the maneuver is a specified travel time (t_{vg}) at whatever speed the two vehicles are traveling.

This approach to Case IIIB has several key advantages. First, it explicitly incorporates the perception-reaction times of both drivers which have not both been considered before in ISD models except in the Connecticut model. Second, it incorporates explicitly the deceleration rate of the major-road vehicle. The model does not rely on poorly defined assumptions about the relationships between the initial speed, average speed, and final speed during the deceleration maneuver; instead, the relationships between these speeds are specified exactly. Third, the final speed of the two vehicles is determined from both the acceleration behavior of the minor-road vehicle and the deceleration behavior of the major-road vehicle. Finally, the model is capable of determining ISD requirements based on any specified minimum speed for the major-road vehicle and the sensitivity of ISD to this minimum speed can be determined.

The model formulation is explained below. Although the methodology looks complex analytically, it is actually very simple conceptually. A simple table or chart presenting the ISD requirements determined can be developed for the designer. To simplify sensitivity analyses with the model, the speed-vs.-time and distance-vs.-time relationships in Green Book Table IX-8 have been expressed as regression equations. These are:

$$V_r = 5.754t_a - 0.0753t_a^2 \tag{33}$$

and

$$P = 0.257t_a + 0.781t_a^2 - 0.0665t_a^3 \tag{34}$$

where:

V_r = speed reached by minor-road vehicle at any time t_a during the acceleration maneuver (km/h)
P = distance traveled by the minor-road vehicle from the stopped position to time t_a during the acceleration maneuver (m)
t_a = acceleration time (sec)

The deceleration by the major-road vehicle is represented by the following equation:

$$V_r = V + 3.6a_r(t_a - J_b) \tag{35}$$

where:

V_r = speed reached during deceleration by major-road vehicle (km/h)
V = initial speed of major-road vehicle (km/h) (assumed: V = design speed of major road)
a_r = maximum deceleration rate (m/sec^2)
t_a = acceleration time of minor-road vehicle (sec)
J_b = perception-reaction time of major-road vehicle (sec)

The acceleration of the minor-road vehicle is represented by Equation (33). To determine the time (t_a) at which the speed of the major- and minor-road vehicles become equal, set Equation (33) equal to Equation (35) and solve the resulting quadratic equation for t_a. With this value of t_a, calculate V_r using either Equation (33) or Equation (35).

Next, check whether the calculated speed to which the major-road vehicle has decelerated (V_r) is less than the minimum acceptable speed of the major-road vehicle (min V_r). A revised speed (V_r') is determined as follows:

If $V_r \geq min\ V_r$ *then* $V_r' = V_r$

If $V_r < min\ V_r$ *then* $V_r' = min\ V_r$

If V_r' is higher than V_r, a revised value of acceleration time (t_a') that is larger than t_a should be calculated by solving the following quadratic equation for t_a:

$$V_r' = 3.576t_a' - 0.0468t_a'^2 \tag{36}$$

The distance traveled by the minor-road vehicle during its acceleration maneuver (P) is calculated using Equation (34) or (37), depending on whether t_a or t_a' is the selected acceleration time

$$P = 0.257t_a' + 0.781t_a'^2 - 0.00665t_a'^3 \tag{37}$$

This is equivalent to looking up P in Green Book Table IX-8. The h term is then calculated as:

$$h = P - D - 0.5w_\ell - \frac{\pi R}{2} + 2R - 0.278V_r't_{vg} - L_a - nw_\ell - wm \tag{38}$$

where:

h = distance along the major road traveled by the minor-road vehicle (m)
P = total distance traveled by minor-road vehicle (m)
D = distance from front of Vehicle A when stopped on the

minor-road approach to the edge of the major-road traveled way (m)

w_l = lane width on major road (m)

R = radius of turn of minor-road vehicle (m)

V_r' = final speed of both vehicles at end of maneuver (km/h)

t_{vg} = minimum acceptable vehicle gap time (sec)

L_a = length of minor-road vehicle (m)

n = number of major-road lanes to be crossed in completing the turn (Note: The value of n does not include the lane to be entered; thus n = 0 for a right turn or a left turn onto a one-way roadway; n = 1 for a left turn onto a two-lane two-way roadway; n = 2 for a left turn onto a four-lane two-way roadway, etc.)

w_m = width of median to be crossed in completing the turning maneuver (m) (Note: $w_m = 0$ for a left turn onto an undivided highway or for any right turn; if $w_m > L_a + 2$, then left-turn sight distance should be determined from a stopped position in the median rather than from a stopped position on the minor-road approach.)

The distance traveled by the major-road vehicle (Q) is computed as:

$$Q = 0.278V(J_a + J_b) + \frac{(0.278V_r')^2 - (0.278V)^2}{a_r} + 0.278V_r'\left[t_a - J_b - \frac{0.278V_r' - 0.278V_r}{a_r}\right] \quad (39)$$

where:

J_a = perception-reaction time of minor-road driver

Finally, ISD is determined as:

$$ISD = Q - h \quad (40)$$

This modified AASHTO model has several key advantages:

- The model resolves several of the conceptual and theoretical objectives to the current AASHTO model, such as the lack of explicit consideration of the deceleration rate and perception-reaction time of the major-road driver.
- The model can be readily adapted to consideration of the ISD requirements of trucks. The only changes necessary to evaluate a truck as the turning vehicle are to replace Equations (33) and (34) with acceleration relationships appropriate for trucks and to use the length of the truck as L_a. Acceleration relationships for trucks can be based on the work of Hutton (*16*).
- The model can be readily adapted to determine the sight-distance needs for right turns and for left turns onto highways with any number of lanes and any median width.

Table 19 presents representative sight-distance values in metric units obtained with the modified AASHTO model using the following assumed parameter values:

Deceleration rate for major-road vehicle (a_r)	3.0 m/sec^2 (9.8 ft/sec^2)
Perception-reaction time for minor-road vehicle (J_a)	2.0 sec
Perception-reaction time for major-road vehicle (J_b)	2.0 sec
Minimum acceptable vehicle gap time (t_{vg})	2.0 sec
Length of minor-road vehicle (L_a)	5.8 m (19 ft)
Distance from front of stopped vehicle on minor road to edge of major-road traveled way	3.0 m (9.8 ft)
Major-road lane width	3.6 m (12 ft)

Table 20 presents a table in English units equivalent to Table 19. The data in the tables show that, for example, if a speed reduction by the major-road vehicle from the design speed to 70 percent of the design speed were assumed, for a major road with a 110-km/h (68-mph) design speed, the leg

TABLE 19 ISD Values for Turns from Stop-Controlled Approaches Based on Modified AASHTO Model (Metric Units)

Major-road design speed (km/h)	Sight distance (m) for specified minimum speeds expressed as a percentage of the major-road design speed						ISD for Case IIIB based on current AASHTO model (m)
	30%	40%	50%	60%	70%	85%	
30	65	65	65	65	65	65	65
40	85	85	85	90	90	90	90
50	115	115	115	115	120	125	120
60	150	150	150	150	150	160	160
70	185	185	185	185	190	205	205
80	225	225	225	230	235	255	255
90	270	270	270	275	285	310	310
100	320	320	320	325	335	370	380
110	370	370	370	380	395	440	455

Note: The assumed parameter values on which this table is based are listed in the accompanying text.

TABLE 20 ISD Values for Turns from Stop-Controlled Approaches Based on Modified AASHTO Model (English Units)

Major-road design speed (mph)	Sight distance (ft) for specified minimum speeds expressed as a percentage of the major-road design speed						ISD for Case IIIB based on current AASHTO model (ft)
	30%	40%	50%	60%	70%	85%	
20	215	215	215	215	220	225	230
25	280	280	280	285	285	300	290
30	355	355	355	360	365	380	380
35	440	440	440	440	450	475	470
40	530	530	530	535	545	580	580
45	630	630	630	635	655	700	700
50	740	740	740	745	770	830	840
55	860	860	860	865	895	975	990
60	985	985	985	995	1,035	1,135	1,160
65	1,120	1,120	1,120	1,135	1,185	1,315	1,360
70	1,265	1,265	1,265	1,285	1,345	1,510	1,590

Note: The assumed parameter values on which this table is based are listed in the accompanying text.

of the sight triangle along the major road could be reduced from 455 m (1,490 ft) under current AASHTO policy to 395 m (1,300 ft). However, the tables also make clear that even if the minimum speed were reduced to a value below 70 percent of the major-road design speed, there would not be much further change in the resulting sight-distance criteria. Field studies described in the next section were conducted to verify some of the key parameter values in the model including the minimum acceptable speed of the major-road vehicle.

FIELD STUDY RESULTS

Field studies were conducted to provide data needed to evaluate, and quantify the parameters for, two candidate alternative ISD models for Stop-controlled intersections: the gap-acceptance model and the modified AASHTO model. This section discusses the field study approach; the study sites and data collection periods; the data reduction and analysis; and the implications of the field study findings for each of the alternative models. A more detailed description of the field studies and subsequent data reduction and analysis activities is presented in Appendix F.

Field Study Approach

Field studies were conducted at selected intersections with Stop control on the minor road to evaluate the gap acceptance and modified AASHTO acceleration/deceleration models for ISD Case III. These studies used a combination of videotape recording to document intersection traffic operations (including the durations of accepted and rejected gaps) and traffic sensors (to measure speeds and headways of vehicles on the major road upstream and downstream of the intersection).

The objectives of the field studies were to determine, for vehicles turning left and right onto the major road from a Stop-controlled approach:

- The durations of gaps in major-road traffic that are accepted and rejected by the driver of the minor-road vehicle;
- The speeds, accelerations, and decelerations used by the minor-road vehicle and the following major-road vehicle, when a gap in major-road traffic is accepted by the driver of the minor-road vehicle;
- The amount by which major-road vehicles reduce their speeds to accommodate minor-road vehicles entering the highway; and
- The minimum separation between the minor-road vehicle and the following major-road vehicle at any point during the maneuver.

Figure 21 illustrates typical data-collection setups for the field studies. The data-collection equipment is intended to provide data on the behavior of each minor-road driver turning left or right onto a two-lane major road and the behavior of the following major-road driver who may be forced to slow because of the minor-road driver's maneuver. Two video cameras are used: one to record gap-acceptance data and a second to record vehicle stopping positions (see discussion in Chapter 6) as well as to provide a second view of accepted and rejected gaps. Traffic sensors were installed at intervals of approximately 60 to 120 m (200 to 400 ft) on the major road, both upstream and downstream of the intersection to record vehicle speeds and headways. The sensors provide sufficient information to trace the speed profiles, separation distances, and acceleration/deceleration behavior of the major- and minor-road vehicles. The furthest upstream sensor on the major-road approach was located at a point before the intersection began to influence traffic operations and, whenever possible, before the intersection even came into the view of the major-road driver.

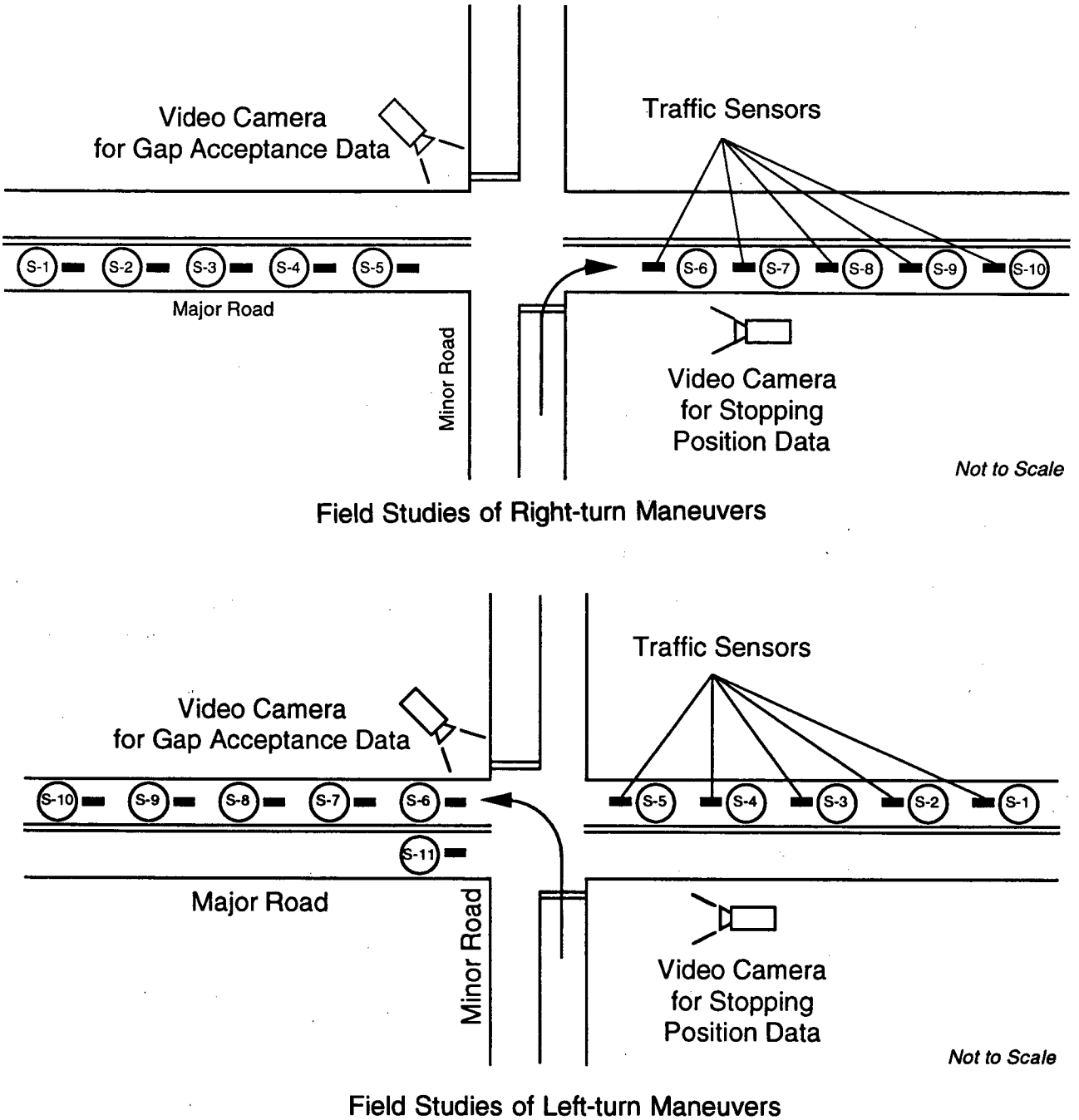


Figure 21. Typical data collection setups for field studies at Stop-controlled intersections.

Study Sites and Data Collection Periods

Field data were collected at a total of 13 Stop-controlled intersections located in three states: Illinois, Missouri, and Pennsylvania. The characteristics of these intersections are summarized in Table 21. The study sites included five intersections with three legs and eight intersections with four legs. At each study site, the major- and minor-road approaches intersected at an angle of approximately 90 degrees. Six of the study sites were located in rural areas, four were located in suburban areas, and three were located in fringe areas (i.e., undeveloped areas near the edge of a metropolitan area on roads that carry substantial commuter traffic).

Each of the study sites had Stop control on the minor road and no control on the major road. Two of the study intersections also had flashing beacons to supplement the Stop control (i.e., flashing red on the minor road and flashing yellow on the major road).

The intersections were selected by reviewing candidate sites suggested by three participating highway agencies: the Illinois Department of Transportation, the Missouri Depart-

TABLE 21 Characteristics of Stop-Controlled Intersections Included in Field Studies

Site number	Area type	Number of legs	Orientation of approach legs studied		Estimated ADT (veh/day)		Available ISD for minor-road driver, m (ft)[a]				Posted speed limit on major-road approach, km/h (mph)		Remarks
							NB/EB approach		SB/WB approach				
			Major road	Minor road	Major road	Minor road	to left	to right	to left	to right	NB/EB	SB/WB	
IL01	Fringe	3	EB/WB	NB	10,600	2,000	Unlimited[b]	Unlimited	–	–	80 (50)[c]	80 (50)[c]	
IL02	Rural	3	EB/WB	SB	9,700	4,700	–	–	370 (1,200)	340 (1,100)	88 (55)[d]	88 (55)[d]	
MO01	Fringe	3	NB/SB	WB	11,200	2,000	–	–	410 (1,330)	440 (1,420)	80 (50)	72 (45)	
MO02	Suburban	4	EB/WB	SB	11,000	–	–	–	310 (1,000)	320 (1,050)	88 (55)	88 (55)	
MO03	Suburban	4	EB/WB	SB	8,000	3,500	–	–	330 (1,060)	450 (1,470)	72 (45)	72 (45)	Flashing beacon
MO04	Suburban	3	EB/WB	SB	10,200	6,600	–	–	180 (600)	Unlimited	56 (35)	56 (35)	
PA01	Rural	3	NB/SB	EB	–	–	270 (880)	80 (250)	–	–	72 (45)	72 (45)	
PA02	Rural	3	NB/SB	WB	–	–	–	–	320 (1,030)	270 (880)	72 (45)	72 (45)	
PA03	Rural	4	NB/SB	WB	13,500	3,900	–	–	140 (440)	90 (280)	72 (45)	72 (45)	
PA04	Rural	4	NB/SB	EB	5,100	3,200	210 (670)	260 (830)	–	–	72 (45)	72 (45)	Flashing beacon
PA05	Suburban	4	NB/SB	EB/WB	3,600	4,800	420 (1,350)	240 (760)	170 (530)	420 (1,350)	72 (45)	72 (45)	
PA06	Rural	4	EB/WB	SB	1,750	3,200	–	–	30 (80)	250 (800)	72 (45)	72 (45)	
PA07	Rural	4	NB/SB	EB/WB	–	–	Unlimited	150 (460)	190 (600)	Unlimited	72 (45)	72 (45)	

[a] Measured from a position on the minor-road approach 6 m (20 ft) from the edge of the major-road traveled way, as prescribed in the current AASHTO policy for ISD Cases IIIB and IIIC.
[b] Unlimited sight distance means at least 600 m (2,000 ft).
[c] Advisory speed of 72 km/h (45 mph) on major road.
[d] Advisory speed of 64 km/h (40 mph) on major road.

ment of Highways and Transportation, and the Pennsylvania Department of Transportation. Each study site was required to have sufficient traffic volumes that gaps in major-road traffic with durations of 10 sec or less occurred with some frequency, at least during the peak hours. Preliminary observations were conducted to ensure that interactions between minor-road vehicles turning left or right onto the major road and following major-road vehicles occurred frequently enough that field studies would provide sufficient data. Finally, each intersection needed to be sufficiently isolated that speed, acceleration, and deceleration data could be collected over lengths of 300 to 450 m (1,000 to 1,500 ft) upstream and downstream of the intersection without substantial interference from adjacent intersections or driveways. This criterion limited the consideration of low-speed urban conditions.

The major road at each study intersection was a two-lane, two-way highway. Multilane highways were not included because, if a minor-road vehicle turns onto a multilane highway, the following major-road vehicle may be able to avoid slowing down by changing lanes. Such maneuvers would substantially limit the ability to measure the typical slowing by major-road vehicles. However, on a two-lane major road, the following road vehicle cannot easily avoid slowing if impeded by a minor-road vehicle turning onto the major road.

All of the intersections studied had good safety records for the particular left- and right-turning maneuvers of interest. None of the study intersections experienced more than 0.7 accidents per year associated with the turning movement of interest, and many of the intersections experienced no accidents in a three- to five-year period associated with that particular turning movement.

The major-road approaches to the study intersections had posted speed limits or advisory speeds ranging from 56 to 72 km/h (35 to 55 mph). Sight distances for ISD Cases IIIB and IIIC, measured from a driver's eye position on the minor road 6 m (20 ft) from the edge of the major-road traveled way ranged from 90 m (280 ft) to essentially unlimited (over 610 m or 2,000 ft). The intersection approach with the most limited sight distance (the westbound approach to Site PA03) had much better sight distance when the minor-road vehicle moved closer to the major-road traveled way.

Table 22 summarizes the dates, times of day, and extent of data collection at each study site. A total of 229.5 hr of traffic operational data were collected in studies over the period from December 1993 to October 1994. Of these studies, 108.5 hr of data were collected for right turns by minor-road vehicles and 121.0 hr of data were collected for left turns by minor-road vehicles. The table identifies the dates and the times of day when data were collected for each approach at each site. The studies were generally performed in 2-hr periods, since 2 hr is the length of a standard videotape; the 229.5 hr of data are included on 118 individual videotapes of gap-acceptance data. In most cases, a total of 4 to 6 hr of data (two to three videotapes of gap-acceptance data) were obtained in a typical data-collection day. (As explained earlier, a separate camera at each intersection was used to document vehicle stopping positions.) The approaches and turning maneuvers studied were selected based on traffic-volume patterns to maximize the number of turning maneuvers of interest observed at particular times of day.

Data Reduction and Analysis

Two major studies were undertaken with the field data. First, the videotapes of traffic operations at the intersections were reviewed to determine the durations of gaps accepted and rejected by each minor-road driver. Second, for a sample of the accepted gaps, the speed profiles of the minor-road vehicle and the following major-road vehicle were traced so that the position of each vehicle at any point in time could be determined; this allowed examination of the acceleration and deceleration behavior of both vehicles. The data reduction and analysis activities for each of these studies are explained below.

In addition, a study of vehicle stopping positions on approaches to Stop-controlled intersections was undertaken at many of the same sites used for the gap-acceptance and acceleration/deceleration studies. The stopping position field study is described in Chapter 6 and Appendix H.

GAP-ACCEPTANCE STUDY

An initial review or screening of each gap-acceptance/rejection videotape was performed to select those tapes that were the best candidates for complete data reduction. Six of the 118 videotapes were excluded because the video camera was misaimed during all or part of the study. Other videotapes were excluded because traffic volumes were lower than desired, so there would be few potential conflicts between major-road vehicles and turning vehicles from the minor road. Based on this screening process, 63 videotapes (or approximately 53 percent of the entire data set) were selected as the highest priority for data reduction. These high-priority videotapes were then reduced and analyzed.

The field dataset includes observations of driver acceptance and rejection of gaps and lags of various lengths. A gap is the time headway between two vehicles on the major road into which a minor-road vehicle may choose to turn. A lag is the portion of a gap that remains when the minor-road vehicle first arrives at the stop line or first begins to move onto the major road. Gap lengths were measured in the study by the elapsed time from the crossing of the centerline of the

TABLE 22 Summary of Data Collection Periods for Field Studies at Stop-Controlled Intersections

Site number	Minor-road approach studied	Turning maneuver studied	Total duration of study (hr)	Study date	Time of day/conditions
IL01	NB	Right turn	2.0	4/13/94	AM peak
IL01	NB	Left turn	4.0	4/13/94	Off peak
IL02	SB	Left turn	2.0	4/14/94	AM peak
MO01	WB	Right turn	4.0	3/9/94	PM peak, off peak
MO01	WB	Left turn	4.0	3/10/94	PM peak, off peak
MO02	SB	Right turn	16.0	3/15/94, 6/24/94, 6/27/94	Off peak, heavy truck volumes
MO02	SB	Left turn	10.0	3/16/94, 6/28/94	Off peak, heavy truck volumes
MO03	SB	Right turn	9.0	3/14/95, 11/2/94	PM peak, off peak
MO03	SB	Left turn	15.0	3/17/94, 3/18/94, 11/3/94	PM peak, off peak
MO04	SB	Right turn	11.0	10/25/94, 10/28/94	PM peak, off peak
MO04	SB	Left turn	12.0	10/26/94, 10/27/94	PM peak, off peak
PA01	EB	Right turn	12.0	5/20/94, 10/14/94	PM peak, off peak
PA01	EB	Left turn	23.5	12/3/93, 5/27/94, 8/23/94, 10/11/94, 10/12/94	PM peak, off peak
PA02	WB	Right turn	11.5	6/8/94, 9/2/94	PM peak, off peak
PA02	WB	Left turn	11.5	6/7/94, 9/23/94	PM peak, off peak
PA03	WB	Left turn	8.0	9/7/94, 9/8/94	Off peak
PA04	EB	Right turn	6.0	9/16/94	Off peak
PA04	EB	Left turn	7.0	9/15/94	Off peak
PA05	EB	Right turn	7.0	9/22/94	Off peak
PA05	EB	Left turn	6.0	9/21/94	Off peak
PA05	WB	Right turn	6.0	9/18/94	Off peak
PA05	WB	Left turn	6.0	9/19/94	AM peak, off peak
PA06	SB	Right turn	6.0	9/28/94	Stopping position data only
PA07	EB	Right turn	12.0	9/39/94, 10/7/94	PM peak, off peak
PA07	EB	Left turn	6.0	10/4/94	Off peak
PA07	WB	Right turn	6.0	10/6/94	AM peak, off peak
PA07	WB	Left turn	6.0	10/5/94	Off peak
		TOTAL	229.5		

intersection by one vehicle until the crossing of the centerline of the intersection by the next.

Acceptance and rejection of gaps and lags were evaluated as follows. When the minor-road vehicle first arrives at the stop line, the driver evaluates the lag represented by remaining portion of the current gap in traffic on the major road in the lane the minor-road driver plans to enter (the near lane for right turns and the far lane for left turns). If the minor-road driver accepted the initial lag and entered the major road, the length of the accepted lag was noted and the consideration of that minor-road vehicle ended. If the minor-road driver rejected the initial lag, then the length of the rejected lag was noted, and the driver then considered each subsequent gap, in turn. If the driver rejected a particular gap, the length of the rejected gap was noted and the driver then considered the next gap. If the driver accepted a gap, the length of the accepted gap was noted and the next minor-road vehicle was then considered. Whenever a gap was accepted, the lengths of both the accepted gap and the accepted lag (to the next major-road vehicle) were recorded.

The only maneuvers considered were those for which there was no interference from other vehicles that could have affected the gap-acceptance behavior of the minor-road vehicle. For example, gaps accepted by right-turning vehicles were excluded if the following major-road vehicle turned right or left at the intersection rather than continuing straight ahead. Gaps accepted by left-turning vehicles were excluded if the following vehicle in the far lane of the major road made a right or left turn; gaps accepted by left-turning vehicles were also accepted if a near-lane vehicle was present on the major road within a travel time 2 sec more than the travel time at the design speed corresponding to the current AASHTO ISD value for the major-road approach. These criteria for interference by other vehicles were purposely made quite conservative to ensure that no interference is present in any of the observations that are included in the analysis.

A total of 4,277 minor-road vehicles were observed turning right onto the major road in the full data set. Of these 4,277 vehicles, a total of 2,758 right-turning vehicles accepted the initial lag that was in progress when they arrived at the stop line on the minor road, and 1,519 minor-road vehicles rejected the initial lag. Of the subsequent minor-road gaps that were considered by the right-turning drivers, 1,758 gaps were rejected and 1,519 were accepted; in other words, sooner or later, each of the 1,519 turning vehicles accepts some particular gap. In summary, a total of 7,554 acceptance/rejection decisions were evaluated by right-turning drivers, resulting in 4,277 acceptances and 3,277 rejections. A total of 1,311 of the acceptance/rejection decisions (or 17 percent of the total decisions) were excluded from consideration because of potential interference from other vehicles with the gap-acceptance/rejection decision. However, a total of 6,243 decisions (3,341 acceptances and 2,902 rejections) provided usable data. These 6,243 usable right-turn decisions included 5,356 right-turn decisions by passenger cars (86 percent), 363 right-turn decisions by single-unit trucks (6 percent), and 524 right-turn decisions by combination trucks (8 percent).

An analysis of the accepted and rejected gaps and lags has been completed using two methods that have been used in previous work for FHWA (*9*). These are the Raff method (*17*) and the logit method (also known as logistic regression) (*18*). Figure 22 illustrates the application of the Raff method to the data for accepted and rejected gaps and lags in right-turn maneuvers by passenger cars; only right-turn maneuvers in which there was no interference from other vehicles with the gap-acceptance/rejection decision are included in the figure. The Raff method involves determination of the cumulative distribution of the percentage of rejected gaps and the complement of the cumulative distribution of the percentage of accepted gaps. The critical gap is considered to occur at the point at which the two distributions cross; in this case, the critical gap is 6.3 sec, as shown in the figure. The critical gap is determined such that the percentage of rejected gaps larger than the critical gap is equal to the percentage of accepted gaps smaller than the critical gap. It should be noted that Figure 22 combines data for the acceptance/rejection of both gaps and lags. The literature indicates that this is an acceptable practice in Miller (*19*) and Kyte et al. ("Capacity Analysis of Unsignalized Intersections," Draft Final Report of NCHRP Project 3-46, 1995, unpublished).

Figure 23 shows the application of logistic regression to the same dataset. Logistic regression is a statistical technique for developing predictive models for the probability that an event (such as the acceptance of a gap) will or will not occur. When logistic regression is applied to the data shown in Figure 22, the following predictive equation is obtained:

$$\ln\left(\frac{P}{1-P}\right) = -4.75 + 0.730X \tag{41}$$

where:

P = probability that a gap of length X will be accepted
X = gap (sec)

Equation (41) is illustrated by the plot in Figure 23. It can be seen in Figure 23 that the critical gap (t_{50}) for which the probabilities of acceptance and rejection are equal is 6.5 sec. This value can also be obtained by substituting P = 0.50 in Equation (41) and solving for the value of X. The logistic regression method has the advantage over the Raff method in that critical gaps can be estimated for any particular probability of acceptance. The results of the logistic regression analyses for right-turn gap acceptance for each site and for each vehicle type (passenger cars, single-unit trucks, and combination trucks) are presented in Tables F-4 through F-6 in Appendix F. The results of both the Raff method and the logistic regression method are summarized in Table 23 for each of the three vehicle types.

In the gap-acceptance data set, a total of 2,388 minor-road vehicles were observed turning left. Of these 2,388 vehicles, 1,104 accepted the initial lag that was in progress when they first arrived at the stop line, and 1,284 rejected the initial lag. The 1,284 drivers who rejected the initial lag evaluated a total of 5,576 subsequent gaps; 1,284 of these subsequent gaps were accepted and 4,292 were rejected. However, in 556 of the 1,284 accepted gaps, the minor-road driver had to wait for one or more near-lane vehicles to clear before accepting the remaining lag. A total of 7,964 acceptance/rejection decisions were observed, including gaps in the far-lane traffic that were "rejected" because a near-lane vehicle was present and lags in the far-lane traffic that were accepted or rejected after a near-lane vehicle cleared. There were usable data (without interference from other vehicles) for a total of 3,526 acceptance/rejection decisions in left-turn maneuvers including 3,311 left-turn decisions by passenger cars (94 percent), 108 left-turn decisions by single-unit

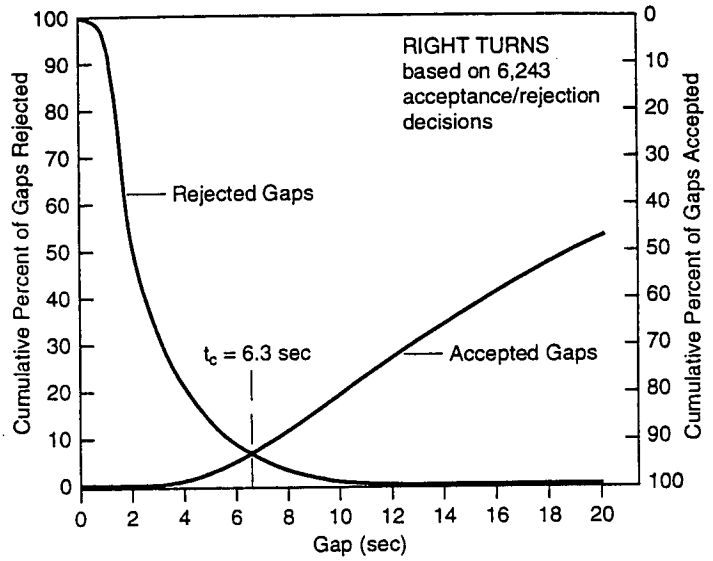


Figure 22. Cumulative distributions of accepted and rejected gaps in right-turn maneuvers by passenger cars.

trucks (3 percent), and 107 left-turn decisions by combination trucks (3 percent). The detailed results of the logistic regression analyses of these data are presented in Tables F-7 through F-9 in Appendix F. Table 22 summarizes the results for left turns from both the Raff method and logistic regression for each of the three vehicle types.

Figure 24 illustrates the distributions for acceptance and rejection of gaps and lags in left-turn maneuvers by passenger cars in accordance with the Raff method. The observed critical gap in the left-turn data from the Raff method is 8.0 sec. For the same dataset, the critical gap (t_{50}) with a probability of acceptance of 50 percent was found from logistic regression to be 9.2 sec, as shown in Figure 25. A comparison shows that the shapes of the logistic regression curves for left- and right-turn maneuvers shown in Figures 23 and 25 differ slightly, and the critical gap of 9.2 sec for left turns by passenger cars is larger than the critical gap of 6.5 sec for right turns by passenger cars. However, when critical gaps are computed giving equal weight to each site, rather than pooling all of the available data without regard to relative number of observations at the various sites, the critical gap for left turns with 50 percent probability of acceptance by passenger cars was found to be 8.2 sec, rather than 9.2 sec.

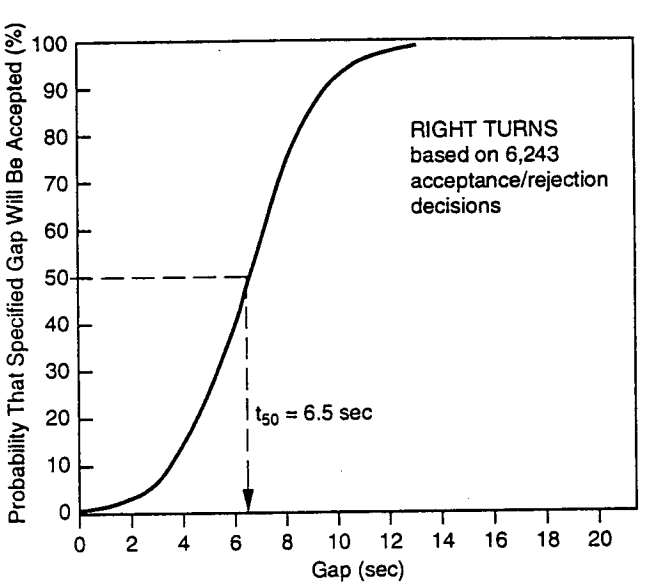


Figure 23. Probability of acceptance of specified gaps in right-turn maneuvers by passenger cars.

Kyte et al. used a more complex analytical approach, known as the maximum-likelihood method, for determining the value of the critical gap for unsignalized-intersection capacity analyses. Kyte et al. raised a concern that logistic regression could, in some cases, provide biased results; however, they also quoted the work of Miller (*19*) which states that where data are normally distributed, the critical gap with 50 percent probability of acceptance from logistic regression provides answers equivalent to the maximum-likelihood method. Therefore, a decision was reached that logistic regression with a 50 percent probability of acceptance was an appropriate method for deriving candidate ISD criteria. This method has been used in previous work by Harwood et al. (*9*) and Fitzpatrick (*20*), among many others.

The results summarized in Table 23 clearly demonstrate that heavy vehicles require longer gaps than passenger cars to enter a major road. This implies that the longer sight distances should

TABLE 23 Critical Gaps Derived from Field Data for Right and Left Turns on a Major Road

Vehicle type	Critical gap (sec)	
	Raff method	Logistic regression
RIGHT-TURN MANEUVERS		
Passenger car	6.3	6.5
Single-unit truck	8.4	9.5
Combination truck	10.7	11.3
LEFT-TURN MANEUVERS		
Passenger car	8.0	8.2[a]
Single-unit truck	9.8	10.8
Combination truck	10.0	12.2

[a] Based on an average giving equal weight to each site.

be provided at intersections where trucks frequently turn onto a highway rather than at locations where few trucks are present. In general, a single-unit truck appears to require critical gaps of 1 to 2 sec longer than a passenger car, and a combination truck appears to require critical gaps approximately 1 to 2 sec longer than a single-unit truck. The truck effect observed in our field studies appears to be slightly larger than the corresponding effect observed by Kyte et al. (see Table 18), which was a difference of 1 sec between passenger cars and heavy vehicles (not distinguished by truck type) for a two-lane major road.

The critical gaps observed for right-turn maneuvers by passenger cars in this study are in close agreement with those observed by Kyte et al., as shown in Table 18.

Kyte et al. recommends a critical gap for right turns of 6.2 sec while, in this research, the Raff method found a critical gap of 6.3 sec and logistic regression found a critical gap of 6.5 sec. The critical gaps for left-turn maneuvers found in this study were just over 1 sec longer than those in Table 18 recommended by Kyte et al. This difference may be explained because the Kyte et al. study included sites with higher traffic volumes, which are likely to put pressure on left-turn drivers to accept shorter gaps.

It is interesting to note that in both the findings of this study and the findings of Kyte et al., left-turning drivers appear to require greater critical gaps than right-turning drivers. This is somewhat surprising because theoretical models, like the current AASHTO model for ISD Case III presented in Equations (26) through (28) and the modified AASHTO model presented above in Equations (35) through (40), suggest that the sight-distance requirements for left and right turns are essentially equal. The observed difference between the critical gaps for right and left turns by passenger cars is 1.7 sec in data from the Raff method (6.3 vs. 8.0 sec), 1.9 sec in the logistic-regression model (6.5 vs. 8.2 sec), but only 0.9 sec in the Kyte et al. data (6.2 vs. 7.1 sec).

It is also interesting to note an analysis of variance by Kyte et al. found the critical gap does not vary with approach speed. This finding supports the notion that a constant value of critical gap, independent of approach speed, can be used in Equations (31) and (32) as the basis for ISD criteria.

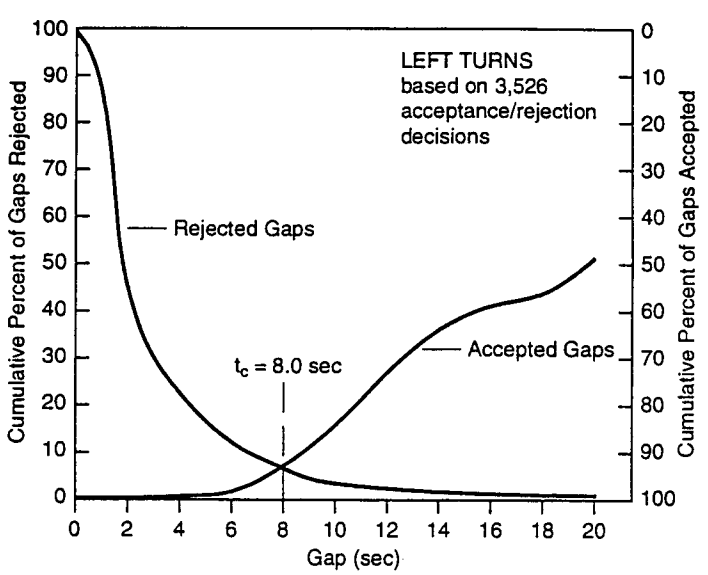


Figure 24. Cumulative distributions of accepted and rejected gaps in left-turn maneuvers by passenger cars.

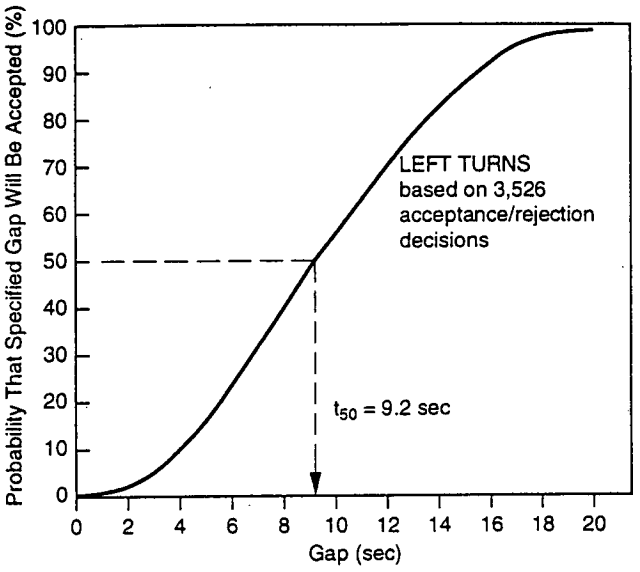


Figure 25. Probability of acceptance of specified gaps in left-turn maneuvers by passenger cars.

Based on the analyses performed, it appears that ISD criteria based on gap acceptance can be developed based on the data in Tables 18 and 23.

ACCELERATION/DECELERATION STUDY

The objective of the acceleration/deceleration study was to determine the distribution of deceleration rates and speed reductions that major-road drivers are willing to use when a minor-road vehicle turns onto the road in from of them. The analysis also evaluated the acceleration behavior of the minor-road vehicle as it enters the major road. The acceleration/deceleration study focused on maneuvers that are critical enough that the major-road driver is likely to find it necessary to reduce speed to accommodate the turning maneuver by the minor-road vehicle.

The candidate maneuvers for the acceleration/deceleration study included all right and left turns for which the accepted gap or lag was 10 sec or less. Turning maneuvers with longer accepted gaps or lags were not considered because there would be little reason for the major-road vehicle to slow down much when the minor-road vehicle is more than 10 sec ahead when it enters the major road; similarly, there would be little reason for the minor-road vehicle to accelerate quickly into traffic when the nearest major-road vehicle is more than 10 sec behind it. Thus, the manuvers selected were those in which there was some incentive for the drivers to use relatively high accelerations and decelerations.

Each candidate maneuver was evaluated by determining the speed of the major-road and minor-road vehicles as they cross each of the sequence of traffic sensors placed on the pavement surface along the major road. These speed profiles were then evaluated to determine the kinematic parameters of interest. All of the computations assume that each vehicle has a constant acceleration or deceleration rate over the 60- to 120-m (200- to 400-ft) interval between each pair of adjacent sensors. The following parameters were determined in the analysis:

- The acceleration or deceleration rate used by each vehicle between each pair of sensors;
- The headway between the major- and minor-road vehicles at each sensor downstream of the intersection, and the minimum headway at any point during the maneuver;
- The location of the major-road vehicle (distance from the intersection) at the instant the minor-road vehicle begins its turning maneuver;
- The maximum speed of the major-road vehicle during the minor-road vehicle's turning maneuver (usually the speed of the major-road vehicle at the instant when the minor-road vehicle begins to turn, but a higher speed may occur in some cases between that point and the intersection);
- The minimum speed reached by the major-road vehicle during the maneuver. Typically, this occurs downstream of the intersection, as the major-road vehicle catches up to the minor-road vehicle. In determining the minimum speed, the speed of the major-road vehicle at each sensor is considered until a point is reached at which the speed of the minor-road vehicle exceeds the speed of the major-road vehicle;
- The speed reduction by the major-road vehicle (the difference between its maximum and minimum speeds, as defined above);
- The speed reduction by the major-road vehicle, expressed as a percentage of its maximum speed;
- The average and maximum acceleration rates of the minor-road vehicle as it completes its turn [the acceleration rates of the minor-road vehicle to speeds of 40 and 64 km/h (25 and 40 mph) were also determined];
- The average and maximum deceleration rates of the major-road vehicle during the maneuver; and
- The separation distance between the major- and minor-road vehicles at each sensor location, and the minimum separation distance during the entire maneuver.

Speed profiles for the major- and minor-road vehicles were obtained for 440 turning maneuvers, including 342 right turns and 98 left turns. Figure 26 shows typical speed profiles for the major- and minor-road vehicles to illustrate how the analysis was conducted. Table 24 presents a summary of the analysis results. Each of the key findings of the acceleration/deceleration analysis is discussed below.

Speed Reduction by the Major-Road Vehicle

In the maneuvers observed involving right and left turns by passenger cars, the maximum speed of the major-road

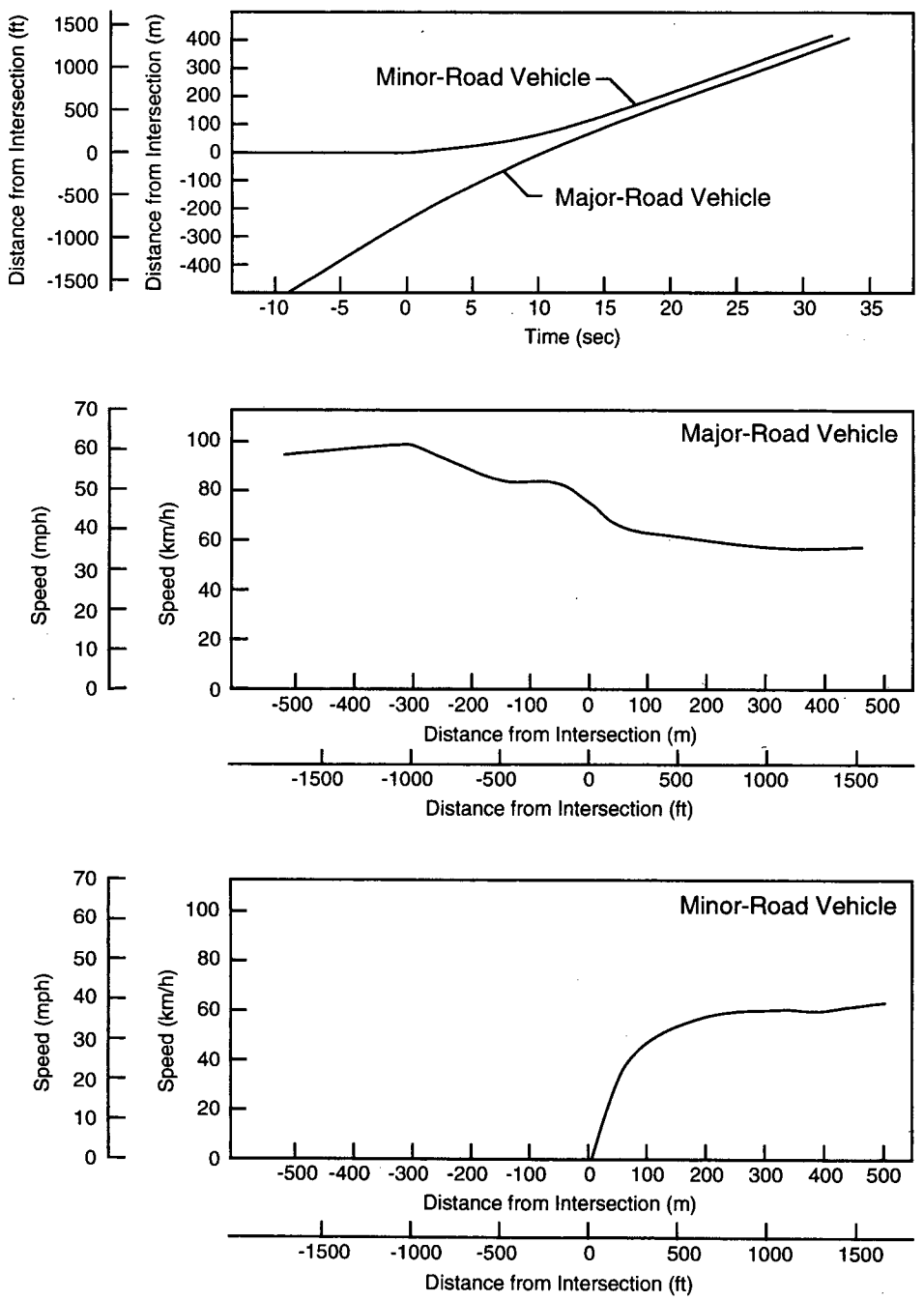


Figure 26. Typical time-space diagram and speed profiles along the major road at a Stop-controlled intersection.

vehicle upstream of the intersection ranged from 42 to 127 km/h (26 to 79 mph), with a mean of 80 km/h (50.1 mph). The minimum speed to which the major-road vehicle decelerated after the minor-road vehicle entered the major road ranged from 16 to 89 km/h (10 to 55 mph), with an average of 53 km/h (33 mph). The speed reductions of individual vehicles ranged from 0 to 84 km/h (0 to 52 mph), with an average speed reduction of 27 km/h (17 mph).

On a percentage basis, the reduction in speed by the major-road vehicle due to the presence of the minor-road vehicle ranged from 0 percent to 80 percent of the maximum upstream speed, with an average speed reduction of 32 percent. The median (50th percentile) speed reduction by the major-road vehicle was 31 percent. By contrast, current AASHTO policy for ISD Cases IIIB and IIIC assumes that the major-road vehicle reduces its speed by only 15 percent, from

TABLE 24 Results of Field Studies of Acceleration/Deceleration Behavior in Turning Maneuvers at Stop-Controlled Intersections

Parameter	Minimum	Mean	Median	85th percentile	Maximum
MAJOR-ROAD VEHICLE					
Maximum speed upstream of intersection, km/h (mph)	42 (26)	80 (50)	79 (49)	98 (61)	127 (79)
Minimum speed after slowing to accommodate minor-road vehicle, km/h (mph)	16 (10)	54 (33)	54 (34)	69 (43)	89 (55)
Reduction in speed to accommodate minor-road vehicle, km/h (mph)	0 (0)	27 (17)	24 (15)	43 (27)	84 (52)
Percentage reduction in speed to accommodate minor-road vehicle	0.0%	32%	31%	51%	80%
Average deceleration rate over distance from maximum to minimum speed, m/sec^2 (ft/sec^2)	–0.04 (–0.12)	–0.68 (–2.23)	–0.53 (–1.7)	–1.13 (–3.7)	–3.23 (–10.60)
Maximum deceleration rate between any pair of adjacent sensors, m/sec^2 (ft/sec^2)	–0.06 (–0.19)	–1.25 (–4.11)	–1.06 (–3.48)	–2.02 (–6.61)	–5.69 (–18.67)
MINOR-ROAD VEHICLE					
Average acceleration rate to a speed of 40 km/h (25 mph), m/sec^2 (ft/sec^2)	0.24 (0.79)	1.49 (4.89)	1.38 (4.51)	2.06 (6.75)	4.23 (13.88)
Average acceleration rate to a speed of 64 km/h (40 mph), m/sec^2 (ft/sec^2)	0.46 (1.50)	1.10 (3.61)	0.98 (3.20)	1.54 (5.04)	4.23 (13.88)
Maximum acceleration rate between any pair of adjacent sensors, m/sec^2 (ft/sec^2)	0.35 (1.14)	1.63 (5.33)	1.51 (4.96)	2.01 (6.89)	4.23 (13.88)

Note: Based on observations of 409 turning maneuvers by passenger cars.

100 percent to 85 percent of the major-road design speed. Figure 27 illustrates the cumulative distribution of the observed percentage reduction in speed by the major-road vehicle.

The field study results show that, on the average, approximately two-thirds of the speed reduction by the major-road vehicle occurs before reaching the intersection, while the remaining one-third of the speed reduction occurs downstream of the intersection.

Deceleration Rate Used by the Major-Road Vehicle

The average observed deceleration rate of the major-road vehicle over the entire distance traveled from its point of maximum speed to its point of minimum speed is 0.68 m/sec^2 (2.2 ft/sec^2). This is a very gentle deceleration rate, far below the maximum deceleration rate of which the vehicle is capable, even on wet pavement. For example, the deceleration rates assumed in the AASHTO policy for stopping sight distance range from 3.9 m/sec^2 (12.8 ft/sec^2) at 30 km/h (19 mi/h) to 2.7 m/sec^2 (9.0 ft/sec^2) at 120 km/h (75 mi/h). The 85th percentile of average observed deceleration rate is 1.1 m/sec^2 (3.7 ft/sec^2).

Major-road drivers are willing to use greater deceleration rates over shorter intervals. Another parameter that was examined was the maximum deceleration rate used by the major-road vehicle over the interval between any pair of adjacent traffic sensors. This maximum deceleration rate had

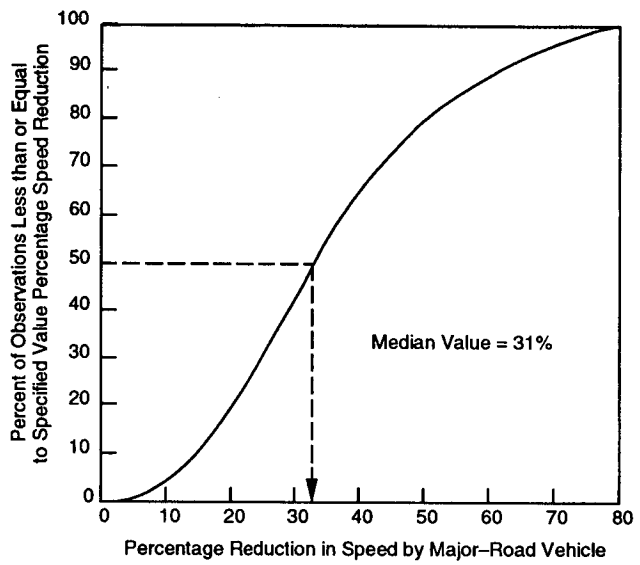


Figure 27. Cumulative distribution of percentage reduction in speed by minor-road vehicle.

a mean of 1.3 m/sec^2 (4.1 ft/sec^2) and an 85th percentile value of 2.0 m/sec^2 (6.6 ft/sec^2).

Acceleration Rate Used by the Major-Road Vehicle

The acceleration profile of the major-road vehicle was also evaluated. The observed average acceleration rate over the speed range from 0 to 40 km/h (0 to 25 mi/h) was 1.49 m/sec^2 (4.9 ft/sec^2) for passenger cars, which agrees almost exactly with the comparable acceleration rate of 1.46 m/sec^2 (4.8 ft/sec^2) assumed in AASHTO policy. The observed average acceleration rate of the minor-road vehicle over the speed range from 0 to 64 km/h (0 to 40 mi/h) was 1.10 m/sec^2 (3.6 ft/sec^2) for passenger cars, which is also quite close to the value of 1.25 m/sec^2 (4.1 ft/sec^2) currently assumed in AASHTO policy. These results confirm Green Book Figure IX-33 and Table IX-8 and suggest that no changes are needed in the acceleration rates used to compute the sight distance needed by passenger cars for crossing and turning maneuvers.

Minimum Headway and Minimum Separation Distance

In the maneuvers observed, the calculated minimum headway between the major- and minor-road vehicles ranged from 0.2 to 10 sec. In approximately 28 percent of the observed maneuvers, the minimum headway during the maneuver was less than the 2.0-sec value assumed in current AASHTO policy. This indicates that the design value for t_{vg} of 2.0 sec currently assumed in AASHTO policy is a very conservative choice. The 15th percentile value of t_{vg} is 1.4 sec. The calculated minimum separation distance between vehicles in the maneuvers observed was found to range from 2 to 215 m (7 to 707 ft).

Interpretation of Results

Based on the field-study results, the gap-acceptance model appears superior to the modified AASHTO model for determining the leg of the clear sight triangle along the major road at a Stop-controlled intersection. As explained below, the gap-acceptance results appear to provide a very safety-conservative approach to sight-distance design. By contrast, the field-study results were only partially helpful in refining the assumed values of the parameters for the modified AASHTO model. The field-study results did show clearly that, on the average, major-road drivers slow by about 30 percent (i.e., to 70 percent of their initial speed) when minor-road drivers accept relatively short gaps to enter the major road. However, even in turning maneuvers with relatively short accepted gaps, the field study showed very modest deceleration rates by the major-road vehicle, far below those assumed in other geometric design policies such as SSD. Because the observed maneuvers were not very critical, the acceleration/deceleration study was not very helpful in determining reasonable assumed values of the maximum deceleration rate (a_r) and the perception-reaction times (J_a and J_b) in the modified AASHTO model.

In summary, the modified AASHTO model still appears to the authors to be a valid approach—conceptually superior to the current AASHTO model and other acceleration/deceleration models that have been proposed—but calibration of this model for design would take field studies of many more turning maneuvers that are more critical (closer to emergency conditions) than those observed in the field studies. The time devoted to such field studies would be substantial, because critical maneuvers of this type are very rare. By contrast, the very noncritical nature of the maneuvers observed in the field studies helps to build confidence that gap-acceptance criteria based on observations of these maneuvers provide a very safety-conservative basis for design.

Another key advantage of the gap-acceptance approach is that it is much simpler and easier to understand than either the current AASHTO model for ISD Case IIIB or the modified AASHTO model presented in Equations (35) through (41).

The results of the field studies show that gap acceptance holds great promise as a method for determining ISD criteria at Stop-controlled intersections. Table 25 compares the results of the current study with two other studies that also address gap-acceptance behavior at Stop-controlled intersections. The table shows good agreement among the results; for both left- and right-turn maneuvers the reported results are within a range of approximately 1 sec. Furthermore, both the Lerner et al. (*13*) and Kyte et al. studies found that the critical gap does not vary as a function of approach speed, which

TABLE 25 Comparison of Gap Acceptance Field Study Results with Results from Other Studies

	Critical gap (sec)			
	Current study			
Maneuver	Raff method	Logistic regression	Lerner et al. (*13*)	Kyte et al.
Right turn from minor road	6.3	6.5	7.0	6.2
Left turn from minor road	8.0	8.2	7.0	7.1

Note: Based on data for turns from Stop-controlled intersections onto a two-lane major road.

supports the concept of using a single critical gap across all design speeds as postulated in Equation (32).

In deciding what value of critical gap to recommend, it seemed evident that geometric design criteria should be more conservative than operational criteria, such as those presented in the *Highway Capacity Manual* (*14*). It also appeared desirable to maintain the current equality between sight distances for left and right turns. The field data from both the current study and the Kyte et al. study indicate that drivers are more conservative in making left turns than in making right turns, even though theoretical models indicate that the sight-distance requirements for both turning maneuvers are essentially the same. Therefore, based on the available data, a critical gap of 8.0 sec appears appropriate for use in sight-distance design for turns by passenger cars. This critical gap is in close agreement with observed driver behavior for left turns and will provide an additional margin of safety of 1.0 to 1.8 sec in right turns.

The acceleration/deceleration-study results provide a still further indication that the field-study results provide a very conservative basis for design. The observed 8.0-sec critical gap was achieved with very modest deceleration rates by major-road vehicles. The observed 85th percentile deceleration rate over the entire deceleration maneuver was 1.2 m/sec^2 (3.7 ft/sec^2), which is only one-third to one-half the deceleration rates that are assumed in SSD, and which can be achieved in an emergency even with poor tires on a poor, wet pavement.

Since the 8.0-sec critical gap, as measured in the field, includes slowing by the major-road vehicle, it is not appropriate for direct use in Equation (32). The equation assumes that the major-road vehicle travels at a constant speed equal to the design speed of the major road. A computation was made of a constant-speed travel time equivalent to the 8.0-sec travel of the decelerating vehicle. When the decelerating vehicle is 8.0 sec from the intersection, the time required for a vehicle to travel from that position to the intersection at a constant speed equal to the initial speed of the vehicle is approximately 7.5 sec. Therefore, it is recommended that the critical gap used in Equation (32) for sight-distance design should be 7.5 sec for right turns by passenger cars onto any major road and for left-turns by passenger cars onto a two-lane, two-way major road.

It should be noted that the recommended gap-acceptance values have been based on the critical gap with a probability of acceptance by the minor-road driver of 50 percent and not on the critical gaps with higher probability of acceptance that have been computed in Appendix F. This appears to be appropriate because, with the gap-acceptance concept for sight distance, as in the real world, it is not the minor-road driver, but the following major-road driver, who has the key responsibility for avoiding a collision. The most critical situation that should occur under the recommended sight-distance criteria is as follows:

(1) A minor-road driver stopped and waiting to turn onto the major road sees no potentially conflicting vehicle at an intersection with sight distance designed in accordance with the 7.5-sec gap acceptance criterion;
(2) The minor-road driver then proceeds to turn onto the major road; and
(3) At just the moment when the minor-road driver begins to turn, a major-road vehicle appears in view.

The field-study results provide evidence that the drivers of major-road vehicles can avoid collisions with vehicles that turn onto the major road by reducing speed by 15 to 50 percent using very modest deceleration rates. Such maneuvers occur regularly in the real world and the field-study results support a recommended design policy based on real-world driver behavior.

The gap-acceptance model contains no explicit term for perception-reaction time, but the model does include the perception-reaction requirements of the drivers of both the major- and minor-road vehicles. As illustrated by the example presented above, the perception-reaction time of the minor-road driver occurs before the 7.5-sec gap begins; the gap-acceptance criterion provides a minor-road driver (who sees no approaching vehicle and enters the major road accordingly) with at least 7.5 sec of clear road at the time the vehicle enters the roadway. The perception-reaction time of the major-road driver is part of the 7.5-sec critical gap; the major-road driver will use a portion of that 7.5-sec period to recognize that the minor-road vehicle has entered the highway and to begin to take action in response, if necessary.

Right turns from Stop-controlled intersections onto multilane highways should not require critical gaps that are longer than right turns onto a two-lane highway, assuming that the right-turning driver enters the right lane of the multilane highway. Turns onto a multilane highway have less potential for collision than turns onto a two-lane highway because the approaching driver on the multilane highway may often be able to change lanes to avoid the conflict. Thus, it is not apparent to the authors why the Kyte et al. study found longer critical gaps for right turns onto multilane highways than onto two-lane highways (see Tables 17 and 18).

For left turns from a Stop-controlled intersection onto a multilane undivided highway, rather than a two-lane highway, the modified AASHTO model in Equations (35) through (40) suggests the additional sight distance required is equal to the lane width of 3.6 m (12 ft). However, the field observations of Kyte et al. suggest that drivers prefer critical gaps that are 0.4 sec longer for left turns onto multilane highways than onto two-lane highways. Therefore, based on rounding of the observed critical-gap value, an allowance of an additional 0.5-sec gap per additional lane to be crossed in a left turn by a passenger car onto a multilane highway is recommended. This allowance of 0.5 sec per additional lane to be crossed is consistent with the slope of the passenger-car acceleration curve shown in Figure 12 (i.e., in Green Book Figure IX-33). In applying this criterion to left-turn maneuvers, it is assumed that the left-turning driver will enter the left-most travel lane on the far side of the roadway. Therefore, only the near lanes need be considered in applying this criterion. For example, the critical gap for a passenger car turning left onto a six-lane major road would be 7.5 sec, plus 0.5 sec for each of the two additional near lanes to be crossed, or a total of 8.5 sec.

For Stop-controlled intersections at which substantial volumes of trucks enter the highway, it is recommended that longer critical gaps be used in sight-distance design. Based on the field-study results (as summarized in Table 23 and presented in more detail in Appendix F), the appropriate critical gaps are 10.0 sec for intersections at which turning maneuvers by single-unit trucks predominate and 12.0 sec for intersections at which turning maneuvers by combination trucks predominate. Using the same logic as presented above for the effect of deceleration by the major-road vehicle, these critical gaps should be adjusted to 9.5 sec for single-unit trucks and 11.5 sec for combination trucks for application in Equation (32). The recommended multilane adjustment for left turns by trucks onto multilane highways is 0.7 sec for each additional lane to be crossed, which is consistent with the slope of the acceleration curves for single-unit and combination trucks in Figure 12.

Normally, the crossing maneuver at a Stop-controlled intersection requires less sight distance than the left- or right-turn maneuver, so the crossing maneuver need not be considered in sight-distance design. For example, Kyte et al. shows that the critical gap for a crossing maneuver at a two-way Stop-controlled intersection on a two-lane highway is 6.5 sec, which is less than the 7.5-sec critical gap recommended here for left and right turns. However, a crossing vehicle must cross more lanes than a left-turning vehicle, so additional time is required for crossing these additional lanes. Thus, on multilane highways, the crossing maneuver should be checked to determine whether it is more critical than the left- and right-turn maneuvers. Normally, this will occur only for roadways wider than six lanes.

The basic critical gap for the crossing maneuver should be 6.5 sec for passenger cars, 8.5 sec for single-unit trucks, and 10.5 sec for combination trucks. For each additional lane to be crossed beyond the initial two lanes, the critical gap should be increased by 0.5 sec for passenger cars and 0.7 sec for single-unit and combination trucks.

Other situations in which the crossing maneuver could be a key consideration in the design of sight distance at a Stop-controlled intersection, all of which are quite unusual, are:

- Intersections where the crossing maneuver is the only legal maneuver;
- Intersections with major-road traffic from the left at which left turns are permitted but right turns are not, with respect to sight distance to the left for crossing the near lanes;
- Intersections with substantial upgrades on the departing leg of the crossroad, which would slow crossing vehicles (particularly trucks) but not turning vehicles (see Figure 19); and
- Intersections at which substantial truck volumes make the crossing maneuver, but not left or right turns.

RECOMMENDATIONS

This section summarizes the recommended sight-distance policy for Stop-controlled intersections and the rationale for that policy. A draft of an intersection sight-distance policy for potential incorporation in a future edition of the AASHTO Green Book is presented in Appendix J.

Recommended Policy for Sight Distance to the Intersection

The driver on the minor road approaching a Stop-controlled intersection should have a view of the intersection and the Stop sign from a distance sufficient to stop before reaching the intersection. This is normally ensured by the provision of SSD along the minor road. However, where a sufficient view of the intersection is not available or where the Stop sign may be obscured, an advance warning sign (i.e., Stop Ahead) should be considered.

Similarly, the driver on the major road needs a view of the intersection sufficient to slow (or, in an emergency, to stop) if a minor-road vehicle crosses or enters the highway. This normally is ensured by the provision of SSD along the major road. However, where the presence of the intersection is not

apparent, an advance warning sign (e.g., the T-intersection or crossroad-symbol sign possibly with an advisory speed plate) should be considered.

Recommended Sight-Distance Policy for the Leg of the Departure Sight Triangle Along the Minor Road at Stop-Controlled Intersections

The leg of the sight triangle along the minor road (a) should have a length of 4.4 m (14.4 ft), as explained in Chapter 6.

Recommended Sight-Distance Policy for the Leg of the Departure Sight Triangle Along the Major Road at Stop-Controlled Intersections

Clear sight triangles like those shown in Figure 28 should be provided in each quadrant of each Stop-controlled intersection. The leg of the sight triangle (b) along the major road should be equal to the distance traveled at the design speed of the major road in the time shown in Table 26. This distance can be computed with Equation (32). The times shown in Table 26 are based on observed gap-acceptance behavior at Stop-controlled intersections. As shown in the table, slightly longer gaps and, thus, slightly longer sight distances are recommended for left turns onto multilane, two-way highways. The adjustment for left turns onto multilane two-way highways involves adding an additional 0.5 sec for passenger cars or an additional 0.7 sec for trucks for each additional lane to be crossed beyond the one lane that would need to be crossed on a two-lane highway. Only the near lanes (carrying traffic from the left) are considered in applying this criterion. For example, the sight distance for a left turn by a passenger car onto a six-lane, two-way highway would be based on a critical gap of 8.5 sec, which consists of the 7.5-sec value that would be appropriate for a left turn onto a two-lane, two-way highway plus 0.5 sec for each of the two additional near lanes that must be crossed in making the left turn.

Any median that is at least 3.6 m (12 ft) in width should be considered in determining the number of lanes to be crossed. For example, a 7.2-m (24-ft) median should be counted as equivalent to two additional lanes to be crossed. However, if the median is wide enough to store the design vehicle with at least 1 m (3.1 ft) clearance at each end, no multilane-highway adjustment is necessary and the travel times as shown in Table 26 should be used without adjustment to determine the sight distance needed to turn left from the median onto the far lanes of the major road.

TABLE 26 Recommended Travel Times for Determining Sight Distance for Left and Right Turns onto the Major Road at Stop-Controlled Intersections

Vehicle type	Travel time (sec) at design speed of major road
Passenger car	7.5
Single-unit truck	9.5
Combination truck	11.5

Note: For left turns onto two-way highways with more than two lanes, add 0.5 sec for passenger cars or 0.7 sec for trucks for each additional lane to be crossed.

Recommended Sight-Distance Policy for the Crossing Maneuver at Stop-Controlled Intersections

The departure sight triangle for the crossing maneuver is generally smaller than the departure sight triangle for the left- and right-turn maneuvers. The recommended travel times (critical gap) used to determine sight distances for crossing two-lane highways are shown in Table 27. The adjustment for crossing maneuvers on multilane highways involves adding 0.5 sec for passenger cars and 0.7 sec for trucks for each additional lane to be crossed beyond two lanes. For example, the sight distance for a passenger car to cross a six-lane highway would be based on a critical gap of 8.5 sec, which consists of the 6.5-sec critical-gap value shown in Table 27 plus 0.5 sec for each of the four additional lanes to be crossed. Normally, the crossing maneuver will become a

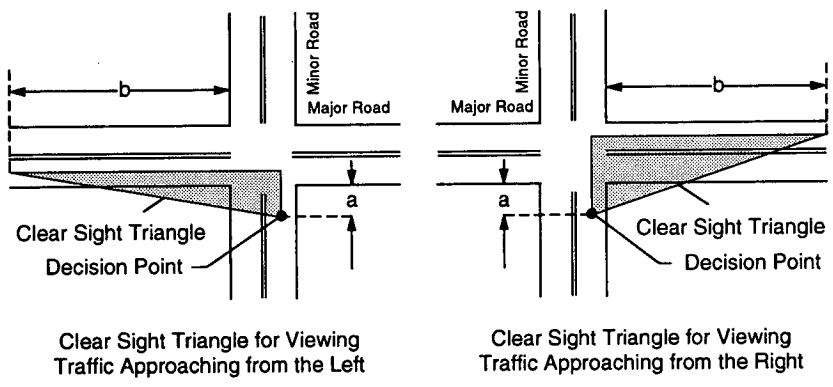


Figure 28. Departure sight triangles for Stop-controlled intersections.

TABLE 27 Recommended Travel Times for Determining Sight Distance for Crossing the Major Road at a Stop-Controlled Intersection

Vehicle type	Travel time (sec) at design speed of major road
Passenger car	6.5
Single-unit truck	8.5
Combination truck	10.5

Note: For crossing a major road with more than two lanes, add 0.5 sec for passenger cars or 0.7 sec for trucks for each additional lane to be crossed.

critical design consideration only for highways with more than six lanes. All lanes in both directions of travel are normally considered. However, any median that is at least 3.6 m (12 ft) wide should be considered in determining the number of lanes to be crossed. For example, a median that is 7.2 m (24 ft) wide should be counted as two additional lanes to be crossed. However, if the median is wide enough to store the design vehicle with at least 1 m (3.1 ft) to space at each end, then the crossing maneuver of each roadway of the divided highway should be evaluated separately.

Discussion of Recommendations

Gap acceptance provides a very safety-conservative approach to the sight-distance requirements of Stop-controlled intersections. The inherent conservatism of the recommended values can be demonstrated in four ways.

- First, the recommended critical gaps are longer than the gaps used for other applications, such as in the procedures of the *Highway Capacity Manual* (*14*) for operational analysis of two-way Stop-controlled intersections.
- Second, the observed 85th percentile deceleration rates of the major-road drivers in the field studies from which the gap-acceptance values were derived are very modest; they are only one-third to one-half of the deceleration rates assumed in SSD design, which are themselves extreme only for vehicles with poor tires on a poor, wet pavement. Thus, at intersections designed in accordance with the recommended gaps, the driver of the major-road vehicle has adequate reserve deceleration capability to respond to occasional misjudgments by one or more of the involved drivers.
- Third, the same gap-acceptance values have been recommended for both right and left turns, although the field studies would support gaps for right turns that are 1.0 to 1.8 sec less than for left turns; this results in very conservative sight-distance values for right turns.
- Finally, all field study sites at which gap-acceptance behavior was observed have good safety records. None of the study intersections experienced more than 0.7 accidents per year associated with the turning movement of interest, and many of the intersections experienced no accidents in a three- to five-year period associated with that particular turning movement. Thus, it can be documented that the intersections that provide the basis for the recommended gap-acceptance values operate very safely. It should also be noted that the recommended policy is similar to those already used by two state highway agencies and a number of local agencies.

Table 28 compares the recommended sight-distance values for Stop-controlled intersections with those presented in current AASHTO policy. The table makes clear that, except at very low speeds, the recommended values constitute a substantial reduction in sight-distance requirements. This reduction is appropriate both because the recommended values are well supported by actual field data, as explained above, and because the current AASHTO values are so long that they are impractical at many real-world intersections. Thus, the recommended values should provide a more practical basis than current policy for determining sight distance needs at Stop-controlled intersections. Finally, the recommended policy also provides sight-distance values for right- and left-turns by trucks at Stop-controlled intersections for application at intersections with substantial numbers of turning trucks. Figure 29 illustrates the recommended sight-distance values for right and left turns by passenger cars and trucks.

Incorporation of Changes to the SSD Model Recommended by NCHRP Project 3-42

Fambro et al. ("Determination of Stopping Sight Distances," Final Report of a forthcoming NCHRP project) has recommended several changes to AASHTO SSD policy in NCHRP Project 3-42. The recommended ISD policy for Stop-controlled intersections presented above is not based on current SSD policy and, therefore, is not affected by the Fambro et al. recommendations.

TABLE 28 Recommended Sight Distances for Left and Right Turns onto the Major Road by Passenger Cars at Stop-Controlled Intersections in Comparison to Current AASHTO Policy

Major-road design speed (km/h)	Recommended sight distance[a] (m)	Sight distance for Case IIIB in current AASHTO policy[b] (m)
30	65	65
40	85	90
50	105	120
60	125	160
70	150	205
80	170	255
90	190	310
100	210	380
110	230	455

[a] Based on 7.5 sec of travel time at the major-road design speed.
[b] Based on Equations (26) through (28).

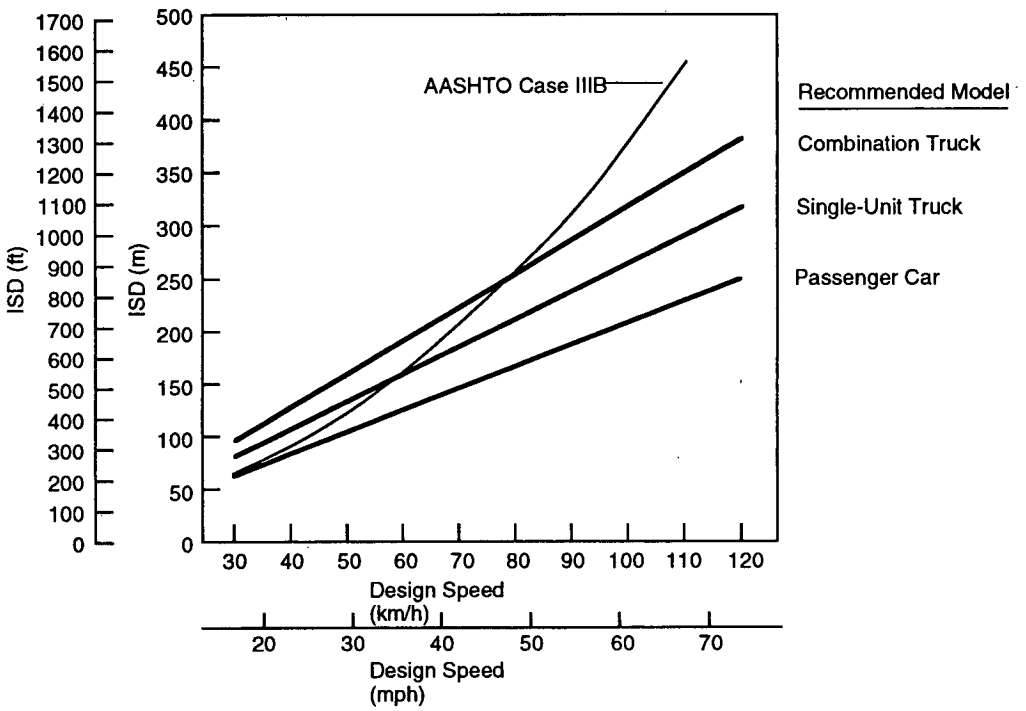


Figure 29. Recommended sight distances along the major road for turning maneuvers at Stop-controlled intersections.

CHAPTER 4

EVALUATION OF ISD POLICY FOR INTERSECTIONS WITH YIELD CONTROL ON THE MINOR ROAD

This chapter presents an evaluation of ISD policy for intersections with Yield control on the minor road.

CURRENT AASHTO POLICY

ISD Case II addresses the sight distance requirements for vehicles on minor-road approaches controlled by Yield signs. Figure 30 illustrates the clear sight triangle used by AASHTO to address this situation. As for uncontrolled intersections, sight distance for Yield-controlled intersection is based on an approach sight triangle. AASHTO policy for ISD Case II is based on the assumption that drivers on the Yield-controlled approach should have sufficient sight distance to stop, if necessary, before reaching the intersection.

AASHTO policy specifies that the sight distance for the driver on the minor road must be sufficient to allow the driver to see a vehicle on the major roadway approaching from either the left or the right and then, through perception, reaction, and braking time, bring the vehicle to a stop prior to reaching the intersecting roadway. Where sufficient sight distance is not available for the driver of the vehicle on the minor road, it may be necessary to have a posted speed reduction on the minor road as it approaches the major cross-road or to use a more positive form of control such as a Stop sign. ISD Case III (Stop Control on Minor Road) should also be provided in addition to ISD Case II so that any vehicle on the minor road that is forced to stop by major-road traffic has sufficient sight distance to proceed.

In a typical case, such as that shown in Figure 30, the speed of Vehicle A is known, as are the distances from the respective vehicle paths to a corner sight obstruction. If Vehicles A and B first sight each other when Vehicle A is at distance d_a from the intersection, then distance d_b can be determined as:

$$d_b = \frac{ad_a}{d_a - b} \tag{42}$$

The critical speed V_b is the speed for which the SSD is d_b. Equation (42) is based on geometric relationships for similar triangles. This critical speed could be used, where necessary, in establishing an advisory speed on the major road. Thus, the current AASHTO policy does not explicitly establish the length of the leg of the clear sight triangle along the major road. However, by implication, the recommended length of the leg of the sight triangle along the major road is equal to SSD for the major-road design speed because, otherwise, an advisory speed would be needed on the major road.

SSD is specified in the AASHTO Green Book as a function of perception-reaction time and braking time. The SSD criteria in the 1994 AASHTO Green Book are based on the following equation:

$$SSD = 0.278t_{pr}V + \frac{V^2}{254f} \tag{43}$$

where:

SSD = stopping sight distance (m)
t_{pr} = perception-reaction time (sec)
V = initial vehicle speed (km/h)
f = coefficient of tire-pavement braking friction

The first term of Equation (43) represents the perception-reaction distance, while the second term represents the braking distance. The coefficient of sliding friction is used by AASHTO in Equation (43) to determine the braking distance for a locked-wheel stop by a passenger car; these braking coefficients generally represent a stop by a vehicle with poor tires on a poor, wet road.

Table 29 presents the AASHTO SSD criteria that appear in Green Book Table III-1. These criteria are based on an assumed perception-reaction time (t_{pr}) of 2.5 sec and the assumed values of speed and coefficient of friction shown in the table. The two values shown in the table for the assumed speed, brake reaction distance, braking distance on level, and stopping sight distance represent minimum and desirable designs based on average running speed and design speed, respectively.

Current AASHTO policy also specifies that departure sight triangles equivalent to those for Stop-controlled intersections should be provided to accommodate left- and right-turn maneuvers by vehicles that stop on the Yield-controlled approach because of the presence of major-road traffic.

CURRENT HIGHWAY AGENCY POLICIES

Table 30 summarizes the design policies used by state and local highway agencies for ISD at Yield-controlled

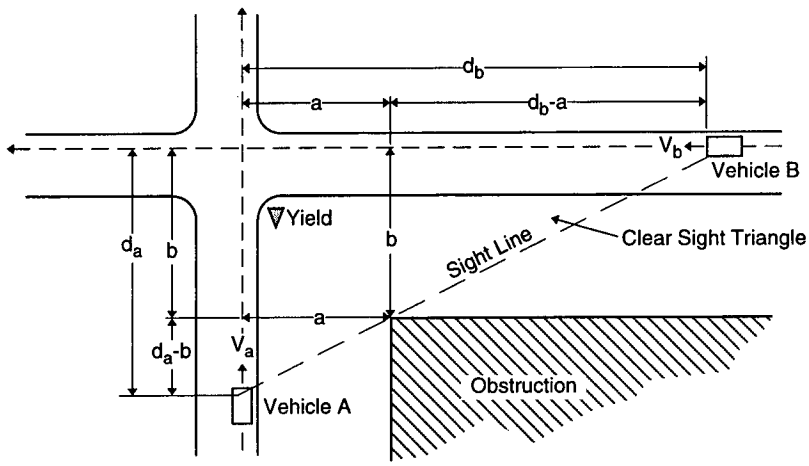


Figure 30. Minimum clear sight triangle used for ISD Case II.

intersections. Over 35 percent of state highway agencies do not consider ISD Case II in their design policies primarily because they do not permit Yield-controlled intersections on the highway system under state jurisdiction. The table shows that, with only four exceptions, all state highway agencies that operate Yield-controlled intersections have design policies based on ISD Case II from either the 1984 or 1990 Green Book (*2,3*). Furthermore, the 1984 and 1990 Green Book policies for AASHTO Case II are identical. It should be noted that the survey on which the table is based was completed prior to publication of the 1994 Green Book (*1*).

Four state highway agencies have their own policies for ISD Case II that differ from AASHTO. These are as follows:

- One state uses a policy that is presented in for *all* intersections and *all* sight distance cases. This policy provides ISD approximately equal to the specified values for ISD Case I and also provides twice the normal SSD along the intersection approaches on the major road.
- One state has a design policy for all ISD cases, including Case II, that is based on the sight distance values for AASHTO Case IIIA.
- One state is considering adoption of a policy for ISD Case II that provides the sight distance required for minor-road vehicles to complete left- and right-turns onto the major road. This policy assumes a reduced vehicle speed of 16 km/h (10 mph) on all Yield-controlled approaches. Since there is no AASHTO SSD value for a design speed of 16 km/h (10 mph), the state uses a calculated SSD value of 15 m (50 ft) as the leg of the sight triangle along the minor road, based on the assumption that f in Equation (44) is equal to 0.25 at 16 km/h (10 mph). The leg of the sight triangle along the major road in this policy ranges from 70 percent to 95 percent of the sight distance for AASHTO Case IIIB.
- One state applies ISD Case II only to Yield-controlled right-turn movements at signalized intersections and ramp terminals. An approach speed of 24 km/h (15 mph) is assumed on the minor road, resulting in an assumed SSD value of 31 m (100 ft).

Table 30 shows that 51 percent of local agencies in urban areas and 35 percent of local agencies in rural areas use

TABLE 29 Braking Coefficients and Stopping Sight Distances Used in Current AASHTO Policy (*1,2*)

Design speed (km/h)	Braking coefficient	Range of SSD (m)	Design speed (mph)	Braking coefficient	Range of SSD (ft)
30	0.40	30-30	20	0.40	125-125
40	0.38	45-45	25	0.38	150-150
50	0.35	60-65	30	0.35	200-200
60	0.33	75-85	35	0.34	225-250
70	0.31	95-115	40	0.32	275-325
80	0.30	115-140	45	0.31	325-400
90	0.30	135-170	50	0.30	400-475
100	0.29	160-205	55	0.30	450-550
110	0.28	180-250	60	0.29	525-650
120	0.28	205-290	65	0.29	550-725
			70	0.28	625-850

TABLE 30 Summary of Highway Agency Design Policies for ISD Case II

Design policy	Number (percentage) of agencies: State highway agencies	Local agencies: Urban	Local agencies: Rural
1990 AASHTO Green Book	18 (38.3)	13 (35.1)	6 (26.1)
1984 AASHTO Green Book	8 (17.0)	6 (16.2)	2 (8.7)
Own state policy	4 (8.5)	– –	4[a] (17.4)
Own local policy	– –	9 (24.3)	3 (13.0)
Don't consider this case	17 (36.2)	9 (24.3)	8 (34.8)
	47	37	23

[a] These local agencies use design policies for ISD Case II based on the policies of the state highway agencies in their states.

the AASHTO policy for ISD Case II. Approximately 24 percent of local highway agencies in urban areas and 13 percent of local highway agencies in rural areas have policies for ISD Case II that differ from AASHTO. Typical local sight-distance policies for Yield-controlled intersections include

- Specified clear sight triangles whose legs vary with approach speed;
- Specified clear sight triangles whose legs do not vary with approach speed. These are typically implemented by local ordinances prohibiting sight obstructions on private property within the specified clear sight triangle;
- Specified clear sight triangle based on safe approach speed greater than 16 km/h (10 mph) determined from the safe approach speed procedure in the Traffic Control Devices Handbook (*4*); and
- Specified clear sight triangle based on sight-distance requirements for a crossing maneuver similar to AASHTO Case IIIA.

Policies on ISD design for Yield-controlled intersections were found in two other countries (see Appendix C). In both Germany and Sweden, the sight distances used as the leg of the clear sight triangle along the minor road are substantially less than the AASHTO SSD values currently used in the United States. The leg of the clear sight triangle used in Germany for passenger cars along the major road is approximately equal to the AASHTO SSD values; the corresponding values used in Sweden are substantially greater than the AASHTO SSD values.

ASSESSMENT OF CURRENT POLICIES

The following discussion presents a critique of the current AASHTO and state and local policies for ISD Case II.

AASHTO Policy

Perception-Reaction Time

The current AASHTO policy assumes that the driver of a vehicle approaching a Yield-controlled intersection will be able to detect a potentially conflicting vehicle on the crossroad and react to that vehicle in 2.5 sec. As in ISD Case I, it is likely that, in some instances, the vehicle on the crossroad will be out of the central part of the visual system of the driver in the approaching vehicle; however, as discussed in Case I, this concern does not account for head movement on the part of the driver. It should also be kept in mind that there is greater sensitivity to motion in the peripheral vision system than there is to recognition of form.

Although the AASHTO ISD policy for ISD Case II makes reference to AASHTO SSD, Case II deals with a different perceptual task (recognizing a vehicle on an intersecting approach rather than an object in the road ahead). Therefore, it should not be simply assumed that the same value of perception-reaction time (2.5 sec) would apply to both tasks. The research that has addressed the perception-reaction time required for ISD Case II is discussed below.

McGee and Hooper (*7*) evaluated the perception-reaction time requirements for ISD Case II. Based on the same theory of subprocesses discussed earlier for Case I, they concluded that the same perception-reaction time values should be used for Case II as for Case I. Thus, McGee and Hooper clearly distinguish between the perceptual tasks in Case II and the different perceptual tasks in SSD but consider the perceptual tasks in ISD Cases I and II to be equivalent. For both Cases I and II, McGee and Hooper recommend the use of a perception-reaction time value of 3.4 sec as appropriate for the 85th percentile of the driving population. However, as discussed in Chapter 2 of this report, the McGee and Hooper estimates appear to be very conservative because they do not consider the possibility of time overlaps or parallel processing of the various sub-processes that were considered.

In contrast, Hostetter et al. (*12*) concluded from a field study that the current value of 2.5 sec is adequate for the Case II perception-reaction time. In addition to their field confirmation of the 2.5-sec value, Hostetter et al. also noted the recent work by Olson et al. (*10*), which confirmed the appropriateness of the 2.5-sec perception-reaction time value for the 95th percentile of the driving population for SSD.

The authors agree that the perceptual requirements of ISD Cases I and II are essentially equivalent. Therefore, as in Case I, we recommend a perception-reaction time of 2.5 sec for use in ISD Case II.

Deceleration Characteristics

The tire-pavement braking coefficients assumed in AASHTO SSD policy are shown in Table 29. These braking coefficients are based on locked-wheel braking by a passen-

ger car and are appropriate for braking by a vehicle with poor tires on a poor, wet pavement. These braking coefficients appear to be appropriate for use in ISD Case II. However, Fambro et al. ("Determination of Stopping Sight Distances," Final Report of a forthcoming NCHRP project) has recommended a change in the braking coefficient assumed in SSD such that SSD will be based on controlled braking by a passenger car driver rather than locked-wheel braking. If the recommended braking coefficient is adopted as AASHTO policy for SSD, then it is recommended that it be incorporated in ISD policy as well.

Recent research for FHWA by Harwood et al. (*9*) reviewed whether the current AASHTO SSD policy was appropriate for trucks. Truck drivers must use controlled braking rather than locked-wheel braking, because trucks can lose control if their wheels lock. Harwood et al. found that longer braking distances and additional SSD are needed for controlled stops by trucks with conventional braking systems. However, the provision of longer SSD for trucks was found to be cost-effective only for sites with very high truck volumes. Furthermore, the provision of antilock braking systems for trucks, which will soon be required by government regulation, should reduce or eliminate the need to provide longer SSD for trucks. Therefore, no special braking coefficients for trucks are recommended for use in determining sight distance at Yield-controlled intersections.

Decision Making by the Minor-Road Driver

The AASHTO Case II model assumes that the minor-road driver will make a choice to stop or continue through the intersection when that driver is at a distance from the intersection equal to SSD. If the minor-road driver sees a potentially conflicting vehicle on the major road from this point, the driver has a sufficient length of roadway available to stop before reaching the intersection. However, if the minor-road driver at the SSD point decides not to stop but to continue at a constant speed equal to the design speed, and a major-road vehicle appears in view shortly thereafter, then a collision may be avoided only if the minor-road driver (or the major-road driver) chooses to adjust speed.

Consider, for example, the intersection of two 80-km/h (50-mph) roadways with minimum available sight distance for ISD Case II. When a minor-road driver is traveling at 80 km/h (50 mph) and is located 145 m or 475 ft (i.e., SSD) from the intersection, that driver cannot yet see a vehicle that is 150 m (490 ft) from the intersection on the major road. That major-road vehicle will appear to the minor-road driver approximately 0.10 sec later, when it is already too late for the minor-road driver to stop before reaching the intersection. In this case, the minor-road driver can avoid a collision by adjusting speed without stopping. However, unlike ISD Case I, it is not reasonable to assume that there is unlikely to be a second vehicle following the first on the major road. Therefore, it is important that drivers on a Yield-controlled approach be able to see any potentially conflicting vehicle on the major road before they pass the last point from which they could stop before reaching the major road.

A minor-road vehicle that does not stop at the intersection can make any of three maneuvers—cross the major road; turn left onto the major road; or turn right onto the major road. The current AASHTO policy is intended to ensure that, if the minor-road vehicle makes one of these maneuvers, the major-road vehicle can avoid a collision by stopping. However, longer sight distances would be required to ensure that the major-road vehicle can avoid a collision by adjusting speed without stopping.

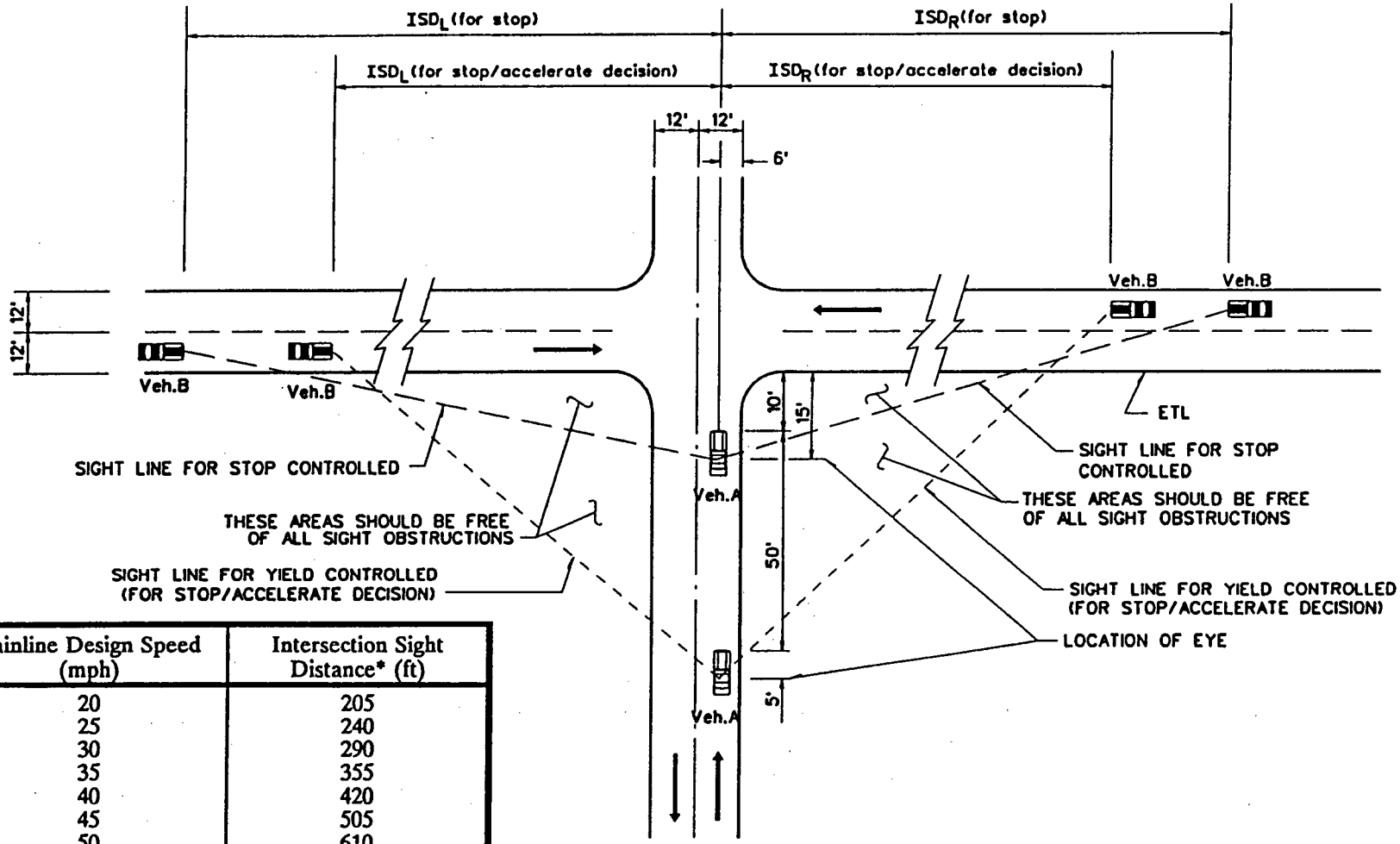
State Policies

Only one state highway agency policy appears to recognize that the sight distance requirements for Yield-controlled approaches should consider the possibility of left- and right-turn maneuvers, as well as stopping or crossing maneuvers. The Oklahoma Department of Transportation is in the process of adopting a new policy for ISD Case II that provides sufficient ISD for a minor-road vehicle to turn left or right onto the major road at a Yield-controlled intersection, as well as to cross the major road or stop. This policy is based on the assumptions that, as drivers approach a Yield-controlled intersection, they typically will

- Slow down as they approach the major road;
- Based on their view of the major road, make a stop/accelerate decision; and
- Either brake to a stop or continue their turning maneuver onto the major road.

This policy is practical because the Oklahoma DOT assumes that the minor-road vehicle will slow to 16 km/h (10 mph) before making the stop/accelerate decision. It is assumed that the minor-road driver makes the stop/accelerate decision when the minor-road vehicle is at a distance from the intersection equal to SSD for a speed of 16 km/h (10 mph), or 15 m (50 ft), plus an allowance of 1.5 m (5 ft) for the setback of the driver's eye from the front of the vehicle.

Figure 31 illustrates both of the clear sight triangles needed for Yield-controlled intersections under the proposed Oklahoma policy. The sight triangle with its vertex 20 m (65 ft) from the intersection along the minor road represents the required sight distance for left- and right-turn maneuvers made without stopping. The other sight triangle with its vertex 4.6 m (15 ft) from the intersection along the minor road represents the required sight distance for left- and right-turn maneuvers made after a stop. The table that appears in Figure 31 shows the required sight distance along the major road (i.e., the leg of the clear sight triangle along the major road).



Mainline Design Speed (mph)	Intersection Sight Distance* (ft)
20	205
25	240
30	290
35	355
40	420
45	505
50	610
55	720
60	835
65	970
70	1125

* These are the ISD criteria for the stop/accelerate decision point (see Section 9.2.4). See Sections 9.2.2 and 9.2.3 for ISD criteria for stop condition.

Figure 31. ISD Case II policy used by Oklahoma DOT to provide sight distance for left and right turns by passenger cars at Yield-controlled intersections.

The sight distances along the major road shown in Figure 31 have been derived by the Oklahoma DOT from the following assumptions:

- The design vehicle is a passenger car.
- The turning vehicle accelerates from 16 km/h (10 mph) to 85 percent of the major-road design speed in accordance with the speed vs. time relationship presented in Green Book Table IX-8.
- The major-road vehicle decelerates from the design speed of the major road to 85 percent of the design speed.
- The gap between the turning vehicle and the major-road vehicle at the end of the maneuver is based on providing 2 sec of travel time at the design speed. (It seems to the authors that this should more appropriately be 2 sec of travel time at 85 percent of the design speed.)
- The driver eye height is 1,070 mm (3.50 ft) and the height of object (an approaching passenger car) is 1,300 mm (4.25 ft).

This approach to Case II, thus, is based on many of the same concepts considered in the AASHTO criteria for ISD Cases IIIB and IIIC. The modified AASHTO model for left and right turns presented in Equations (35) through (40) in Chapter 3 of this report also could be adapted to the Oklahoma DOT approach to Yield-controlled intersections.

The concept used by the Oklahoma DOT appears quite promising in that it is the only existing sight-distance concept proposed for Yield-controlled intersections that recognizes that the sight-distance requirements for left and right turns should be addressed. However, the Oklahoma DOT concept does not address the sight-distance requirements of the crossing maneuver. While it appears reasonable to assume that minor-road vehicles must slow to 16 km/h (10 mph) in making a turning maneuver, it is not clear that drivers would slow this much in making a crossing maneuver. However, the crossing maneuver is a concern at four-leg intersections, but not at three-leg intersections.

ALTERNATIVE ISD MODELS AND METHODOLOGIES

The Oklahoma DOT approach to sight-distance requirements for Yield-controlled intersections draws attention to the point that existing models and methodologies do not address all of the possible maneuvers that can occur at Yield-controlled intersections.

A number of alternative sight-distance concepts for Yield-controlled intersections were evaluated during the research, including

- Alternative models based on adjusting speed, rather than stopping, by the minor-road vehicle;
- Alternative policies that, for any particular intersection, give the designer a choice between a model based on adjusting speed and a model based on stopping by the minor-road vehicle;
- Alternative models based on left- and right-turn maneuvers, like the Oklahoma DOT model, rather than crossing and stopping maneuvers; and
- Alternative models based on safe approach speed, such as that presented in Figure 2-5 in the *Traffic Control Devices Handbook* (*4*), or a modification of it.

However, none of these concepts appeared capable of addressing all of the many maneuver types that can occur at Yield-controlled intersections. The following maneuvers need to be addressed and sight-distance policies should be based on whichever of these models prove to be most critical:

- Driver of crossing vehicle on the Yield-controlled approach sees potentially conflicting traffic on the major road and brings the vehicle to a stop before reaching the intersection.
- Driver of crossing vehicle on the Yield-controlled approach sees no potentially conflicting traffic on the major road and proceeds to cross the major road without stopping.
- Driver of left- or right-turning vehicle on the Yield-controlled approach sees potentially conflicting traffic on the major road and brings the vehicle to a stop before reaching the intersection.
- Driver of left- or right-turning vehicle on the Yield-controlled approach sees no potentially conflicting traffic on the major road and proceeds to enter the major road without stopping.
- Driver of crossing, left-turning, or right-turning vehicle on the Yield-controlled approach, having stopped before entering the intersection because of the presence of traffic on the major road, accelerates from a stop to cross or enter the major road.

In formulating a recommended sight-distance policy for Yield-controlled intersections, it was recognized that the concepts developed in previous chapters for uncontrolled and Stop-controlled intersections can be adapted to address each of the maneuver types listed above. This approach provides a desirable consistency between the concepts recommended for the various different types of ISD and is therefore recommended. The adaptation of the recommended ISD models and methodologies for uncontrolled and Stop-controlled intersections to fit these cases is addressed in the next section.

DEVELOPMENT OF ISD MODELS AND METHODOLOGIES

The following discussion addresses appropriate models of the ISD requirements for each of the maneuvers of interest at Yield-controlled intersections.

Crossing Maneuver: Conflicting Traffic on Major Road

The first maneuver of interest involves a vehicle whose driver intends to cross the major road from a Yield-controlled approach. A model is needed to determine the last point on that approach from which the minor-road driver can stop before reaching the intersection if a vehicle on the major road comes into view. Such a model can be adapted from the model of stopping behavior recommended for use on uncontrolled intersection approaches. The uncontrolled intersection model is based on the AASHTO SSD model, but incorporates the observation that through drivers on approaches to uncontrolled intersections typically reduce their speed on the intersection approach to 50 percent of their midblock running speed.

The same model is potentially applicable to drivers of crossing vehicles on Yield-controlled approaches. However, field observations at several intersections in Kansas City, Missouri, indicate that drivers on Yield-controlled approaches typically reduce their speed to only 60 percent of the midblock running speed (see Appendix G). Figure 32 presents a typical speed-distance plot (speed profile) for a crossing vehicle on a Yield-controlled approach, showing the speed profile that drivers would follow when braking to a stop and when continuing through the intersection.

The decision point on the approach to a Yield-controlled intersection is the last point from which the approaching driver can brake to a stop (with poor tires on a poor, wet pavement, as assumed in SSD) if a major-road vehicle comes into view. As in SSD design and in the recommended policy for uncontrolled intersections, it is assumed that the driver may require 2.5 sec of perception-reaction time to detect a potentially conflicting vehicle and begin to brake. The decision point at the beginning of perception-reaction time may be either to the left or to the right of Point A in Figure 32, depending on the midblock speed (the possible alternative locations for the beginning of perception-reaction time are illustrated in Figure 8).

The distance from the decision point to the intersection represents one leg of the sight triangle for the crossing maneuver. This distance can be computed with Equations (15) to (19), except that it should be assumed that:

$$V_e = 0.60V \tag{44}$$

Table 31 shows the distances from the decision point to the intersection for various design speeds. Where the design speed for the roadway upstream of an intersection is not known, it can be estimated as the 85th percentile speed of traffic at a midblock location away from the intersection.

Crossing Maneuver: No Conflicting Traffic on Major Road

If the driver of a minor-road vehicle passes the decision point and has not yet seen any potentially conflicting vehicles and has not yet begun to brake, the driver is then committed to continue through the intersection. There is not sufficient time remaining for the driver to brake to a stop before reach-

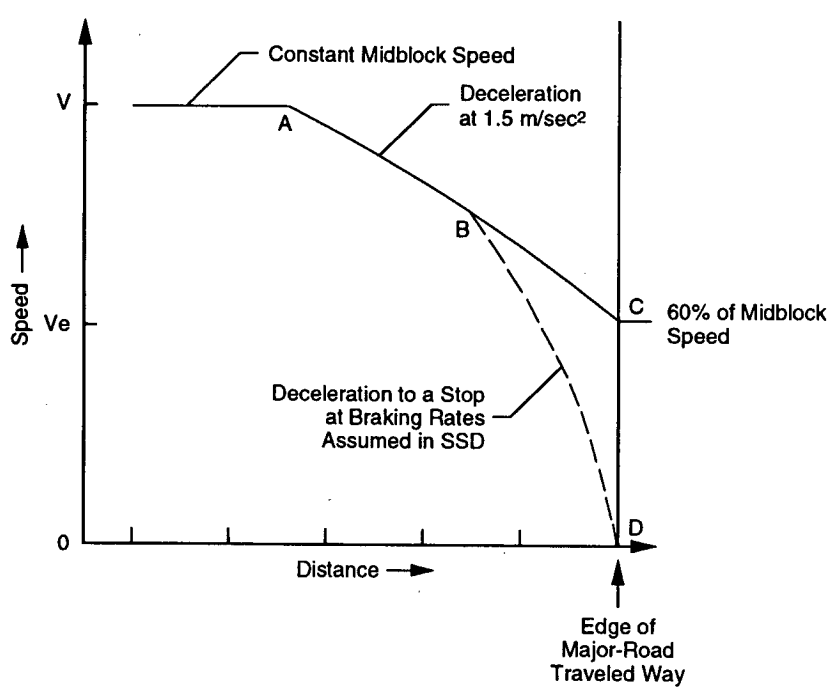


Figure 32. Speed profiles for vehicles intending to cross the major road from a Yield-controlled approach.

TABLE 31 Leg of Sight Triangle Along the Minor Road for Crossing Maneuvers from a Yield-Controlled Approach

Design speed (km/h)	Length of minor-road leg of sight triangle (m)	Current AASHTO SSD criteria[a] (m)	Design speed (mph)	Length of minor-road leg of sight triangle (ft)	Current AASHTO SSD criteria[b] (ft)
30	30	30	20	95	125
40	40	45	25	120	150
50	50	65	30	150	200
60	65	85	35	180	250
70	85	115	40	235	325
80	110	140	45	280	400
90	140	170	50	350	475
100	165	205	55	420	550
110	190	250	60	500	650
120	230	290	65	570	725
			70	675	850

[a] High end of range in 1994 Green Book (rounded).
[b] High end of range from 1990 Green Book.

ing the intersection, at least under the conditions assumed in SSD design (poor tires on a poor, wet pavement). Therefore, it must be assumed that the driver will continue through the intersection. As shown in Figure 32, it is assumed that the driver will continue to slow until a speed equal to 60 percent of the midblock running speed is reached and it is further assumed that the driver will continue at a constant speed equal to 60 percent of the midblock running speed until the vehicle has crossed and cleared the intersection.

Alternatively, it might be assumed that the minor-road vehicle would begin to accelerate again once it entered the intersection; however, the assumption of constant speed is more conservative for passenger cars and, for trucks, it might be unrealistic to assume that the vehicle is able to accelerate.

The travel time from the vehicle position at which the driver's perception-reaction time begins to the point at which the vehicle has cleared the intersection can be determined in English units as:

$$t_a = t_{pr} + \frac{1.47V_e - 1.47V_b}{a_i} \tag{45}$$

$$t_c = t_a + \frac{w + L_a}{1.47V_e} \tag{46}$$

where:

t_a = time for minor-road vehicle to travel from the beginning of perception-reaction time to the point at which the vehicle reaches the intersection (sec)
t_c = time for minor-road vehicle to travel from beginning of perception-reaction time to the point at which the vehicle clears the intersection (sec)
t_{pr} = perception-reaction time (sec)
V_e = speed at which minor-road vehicle would enter the intersection after decelerating (mph) (assumed: V_e = 0.60 V)
V = midblock running speed (mph)
V_b = speed at which braking by the minor-road vehicle begins (mph)
a_i = deceleration rate (ft/sec^2) on intersection approach when braking to a stop by the minor-road vehicle is not initiated (assumed: $a_i = -5.0$ ft/sec^2)
w = width of intersection (ft)
L_a = length of vehicle (ft)

An equivalent expression in metric units is:

$$t_a = t_{pr} + \frac{0.278V_e - 0.278V_b}{a_i} \tag{47}$$

$$t_c = t_a + \frac{w + L_a}{0.278V_e} \tag{48}$$

where the speeds (V_e and V_b) are in kilometers per hour, the distances (w and L_a) are in meters, and the deceleration rate (a_i) is in meters per second per second.

Table 32 shows the computed values of t_a for various minor-road design speeds. The value of t_c is computed from t_a and assumed values of intersection width and vehicle length. The length of the leg of the sight triangle along the major road to accommodate the crossing maneuver is the distance that a major-road vehicle would travel in time t_c at the design speed of the major road.

Turning Maneuver: Conflicting Traffic on Major Road

Vehicles on Yield-controlled approaches that intend to turn left or right at the intersection follow a different speed profile than crossing vehicles. Intersection turning maneuvers are typically made at speeds of approximately 16 km/h (10 mph), so drivers of vehicles on Yield-controlled approaches that intend to turn begin to slow to that speed, if

TABLE 32 Travel Time (t_a) for Minor-Road Vehicle to Reach the Major Road on a Yield-Controlled Approach

Minor-road design speed (km/h)	Travel time for minor-road vehicle, t_a (sec)	Minor-road design speed (mph)	Travel time for minor-road vehicle, t_a (sec)
30	3.4	20	3.5
40	3.7	25	3.7
50	4.1	30	4.1
60	4.7	35	4.6
70	5.3	40	4.9
80	6.1	45	5.3
90	6.8	50	5.9
100	7.3	55	6.5
110	7.8	60	7.1
120	8.6	65	7.5
		70	8.2

necessary, even before they have determined whether any potentially conflicting vehicles are present on the major road. The turning speed of 16 km/h (10 mph) is typically less than the speed to which crossing vehicles typically decelerate on the intersection approach (60 percent of their midblock running speed).

Figure 33 illustrates a typical speed profile of a turning vehicle approaching a Yield-controlled intersection. The figure shows that, since the vehicle is decelerating to reach 16 km/h (10 mph) before reaching the intersection, very little further braking is required to stop if a potentially conflicting vehicle comes into view on the major road. The stopping distance (perception-reaction distance plus braking distance) varies only slightly from 23.1 m (75.8 ft) for midblock speeds of 32 km/h (20 mph) to 23.2 m (76.2 ft) for midblock speeds of 113 km/h (70 mph). Thus, the leg of the sight triangle along the minor road to allow a turning vehicle to stop before reaching the intersection (rounded for design) is only

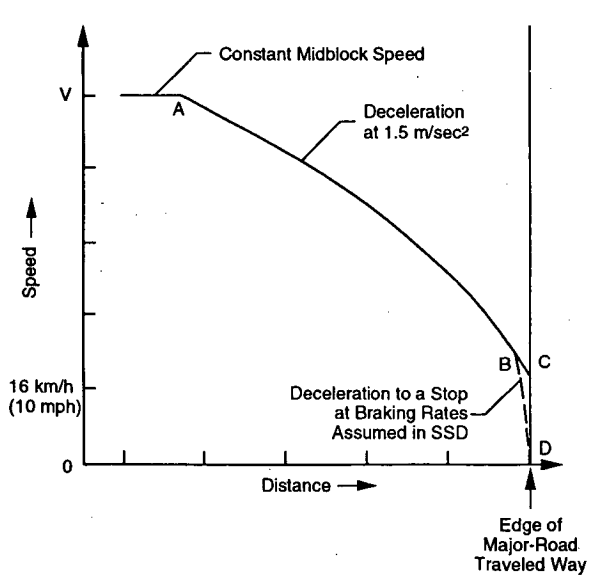


Figure 33. Speed profiles for vehicles intending to turn onto the major road from a Yield-controlled approach.

25 m (80 ft) and does not vary appreciably with the design speed of the minor road.

Turning Maneuver: No Conflicting Traffic on Major Road

The time required for the minor-road vehicle to travel a distance of 25 m (80 ft) to the intersection while decelerating at 1.5 m/sec^2 (5.0 ft/sec^2) is 3.5 sec. This implies that the leg of the sight triangle along the major road (7.5 sec travel time for a turning maneuver by a passenger car, as shown in Table 26) should be increased by 3.5 sec at a Yield-controlled intersection. However, less acceleration time is required by the turning vehicle since it starts from a turning speed of 16 km/h (10 mph) rather than from a stop. The amount of acceleration time foregone by starting from 16 km/h (10 mph) rather than from a stop is 3.0 sec, based on the acceleration times in Green Book Table IX-8. Thus, the travel times used to determine the sight distance along the major road for Stop-controlled intersections given in Table 26 should be increased at Yield-controlled intersection by 3.5 sec and decreased by 3.0 sec, for a net increase of 0.5 sec.

Turning Maneuver: Turning onto the Major Road from a Stop

If a vehicle stops before entering the major road because of the presence of potentially conflicting traffic on the major road, the clear sight triangle for turning onto the major road is exactly the same as the clear sight triangle for Stop-controlled intersections presented in Chapter 3. This sight triangle need not be explicitly considered, however, because it is always smaller than the recommended sight triangle for turning onto the major road without stopping. The sight triangle for crossing the major road from a stop is even smaller than the sight triangle for turning onto the major road from a stop (see Chapter 3) and, therefore, also need not be considered explicitly.

RECOMMENDATIONS

This section summarizes the recommended sight distance policy for Yield-controlled intersections and discusses the rationale for that policy. A draft of an intersection sight distance policy for potential incorporation in the AASHTO Green Book is presented in Appendix J.

Recommended Policy for Sight Distance to the Intersection

The driver on the minor road approaching a Yield-controlled intersection should have a view of the intersection and the Yield sign from a distance sufficient to stop before reaching the intersection. This is normally ensured by the

provision of SSD along the minor road. However, where a sufficient view of the intersection is not available or where the Yield sign may be obscured, an advance warning sign (i.e., Yield Ahead) should be considered.

Similarly, the driver on the major road needs a view of the intersection sufficient to slow (or, in an emergency, stop) if a minor-road vehicle crosses or enters the highway. This is normally ensured by the provision of SSD along the major road. However, where the presence of the intersection is not apparent, an advance warning sign (e.g., the T-intersection or crossroad symbol sign) should be considered.

Recommended Sight Distance Policy for Four-Leg Yield-Controlled Intersections

At four-leg Yield-controlled intersections, the clear sight triangles illustrated in both Figures 34 and 35 should be provided.

Crossing Maneuver

In Figure 34, the length of the minor-road leg (a) of the sight triangle is based on the distances shown in Table 31; the length of the sight triangle along the major road (b) is the distance traveled by a vehicle at the design speed of the major road in the times computed with Equations (47) and (48).

The recommended lengths for the minor-road leg of the sight triangle shown in Table 31 for the crossing maneuver are equal to or slightly less than the current values for the minor-road leg in AASHTO ISD Case II, which are identical to the AASHTO SSD values. This reduction in the length of the minor-road leg of the sight triangle results from the observation that vehicles on Yield-controlled approaches slow to 60 percent of the midblock running speed even when no potentially conflicting vehicle is present. Figure 36 compares the recommended lengths for the leg of the clear sight triangle along the minor road for the crossing maneuver to the recommended values in current AASHTO policy for ISD Case II.

When the design speeds of the major and minor roads are equal, the recommended lengths for the leg of the clear sight triangle along the major road are greater than the values recommended in current AASHTO policy, which are based on the AASHTO SSD criteria. Figure 37 compares the recommended lengths for the leg of the clear sight triangle along the major road for the crossing maneuver to the current AASHTO policy for ISD Case II for this situation with equal sight distances along the major and minor roads. Where the design speed of the minor road is less than the design speed of the major road, the leg of the sight triangle along the major road will be shorter than for the situation in which the design speeds are equal. However, the leg of the sight triangle along the major road to accommodate the crossing maneuver at a Yield-controlled intersection should not be shorter than the design value for crossing the major road from a Stop-controlled intersection, based on Table 27.

Left- and Right-Turn Maneuvers

In Figure 35, the leg of the sight triangle along the minor road (d_a) is 25 m (80 ft), independent of the design speed of the minor road; the length of the sight triangle along the major road is equal to the distance traveled by a vehicle at the design speed of the major road in the travel times shown in Table 26, plus 0.5 sec. As explained earlier, this 0.5 sec adjustment represents the net difference between the increased deceleration time and decreased acceleration time of the minor-road vehicle at a Yield-controlled intersection, as compared to a Stop-controlled intersection.

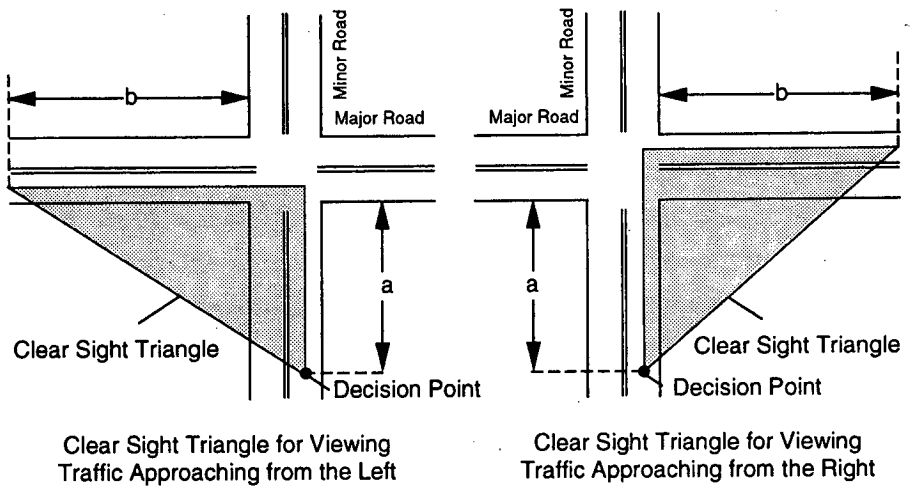


Figure 34. Approach sight triangles for crossing maneuvers at a Yield-controlled intersection.

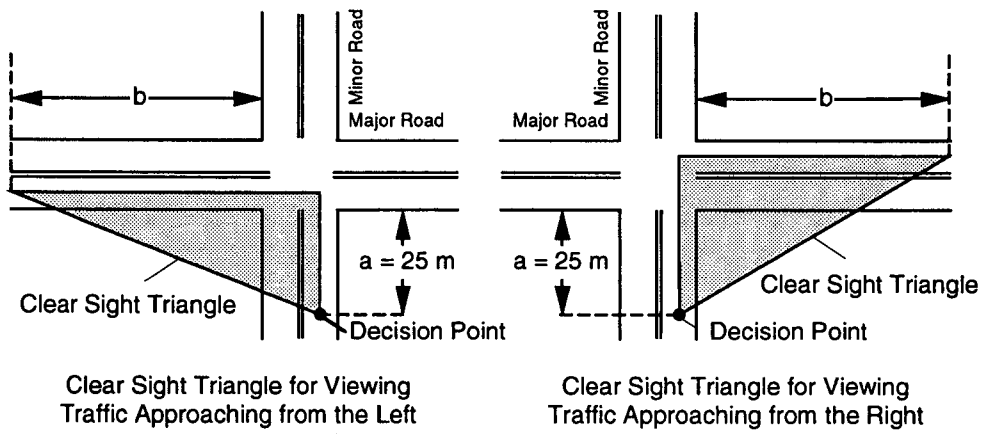


Figure 35. Approach sight triangles for left- and right-turn maneuvers onto the major road at a Yield-controlled intersection.

For a vehicle that intends to turn left or right onto the major road, the 25-m (80-ft) leg of the sight triangle along the minor road is greater than the recommended 4.4-m (14.4-ft) length of the minor-road leg for a Stop-controlled intersection discussed in Chapter 3, but is substantially less than the current AASHTO values for Yield-controlled intersections, which are based on the AASHTO SSD criteria. Figure 38 compares the recommended length of the leg of the clear sight triangle along the minor road for turning maneuvers to the recommended length in current AASHTO policy for Case II.

The recommended values for the leg of the sight triangle along the major road are generally longer than the values in current AASHTO policy for ISD Case II, which are based on SSD. These recommended values are greater than the current AASHTO values because AASHTO does not currently consider the sight distance requirements for turning onto the major road from a Yield-controlled approach without stop-

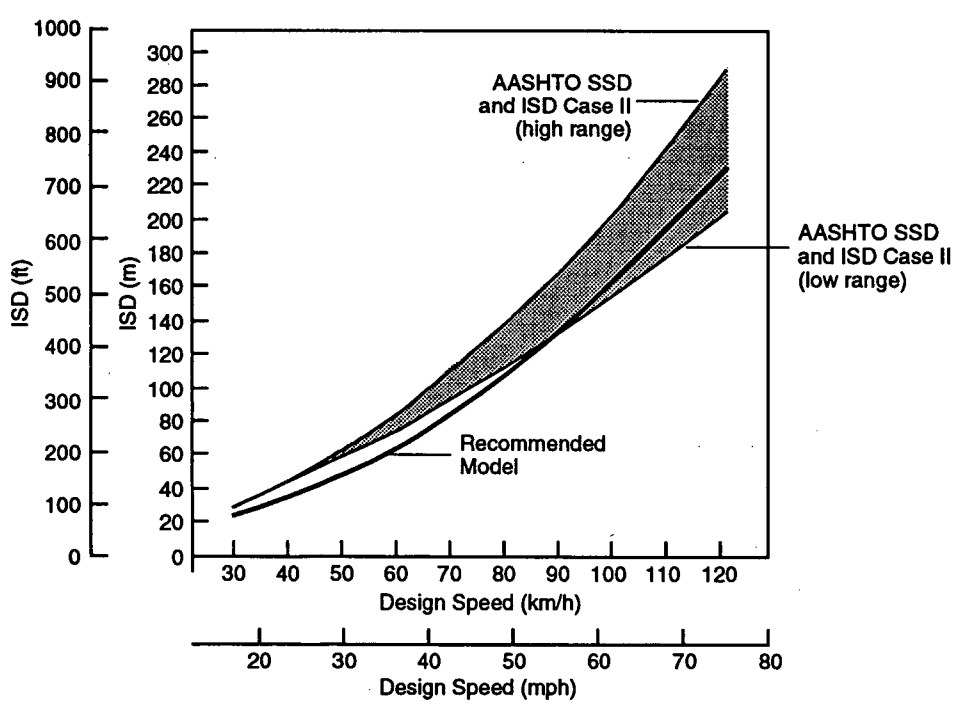


Figure 36. Comparison of recommended values for the leg of the sight triangle along the minor road to accommodate crossing maneuvers at Yield-controlled intersections to current AASHTO policy.

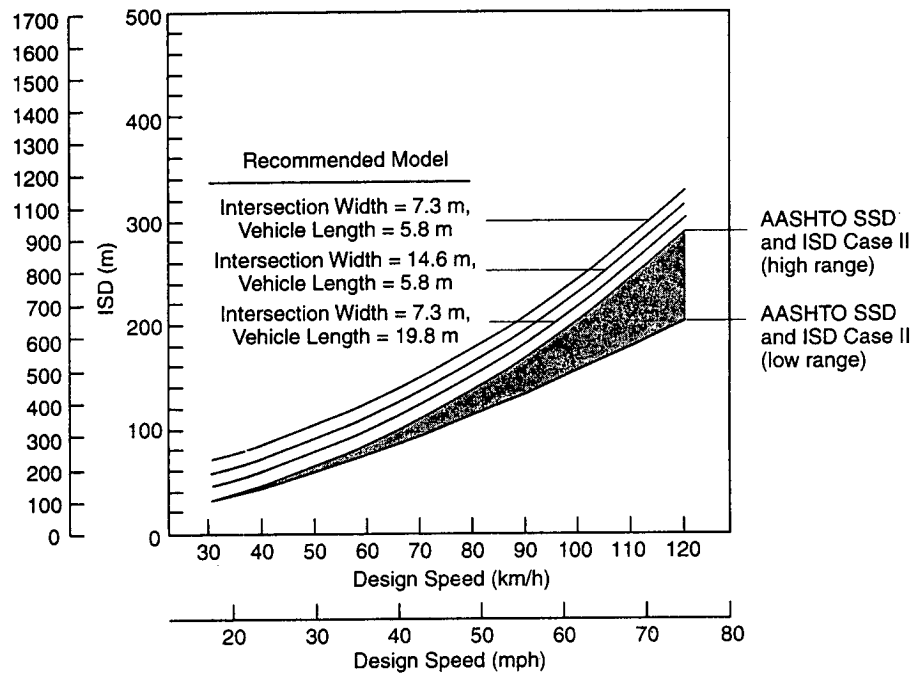


Figure 37. Comparison of recommended values for the leg of the sight triangle along the major road to accommodate crossing maneuvers at Yield-controlled intersections to current AASHTO policy.

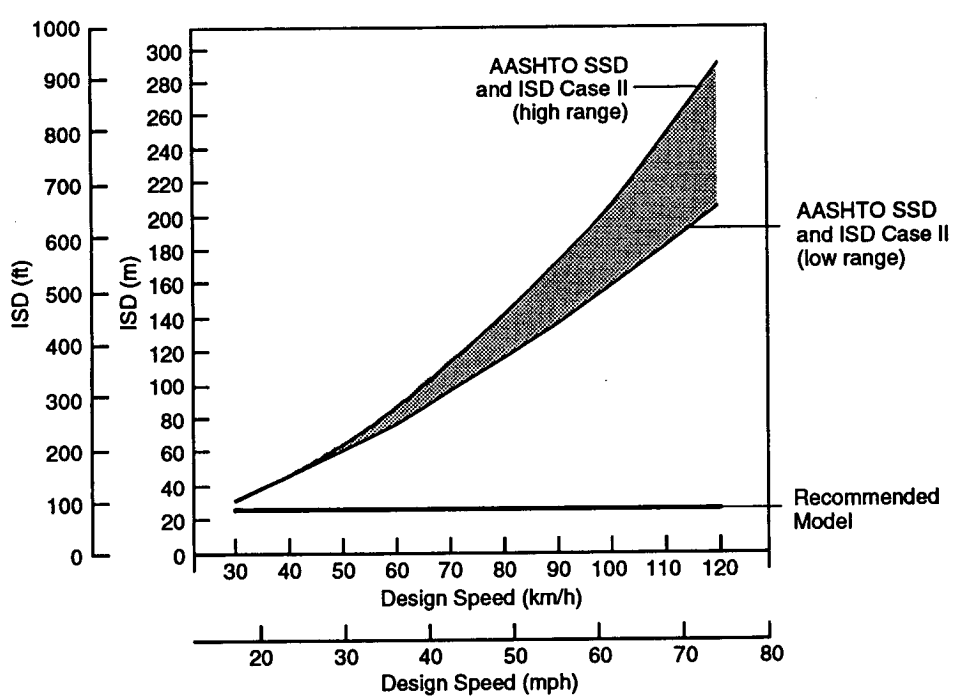


Figure 38. Comparison of recommended values for the leg of the sight triangle along the minor road to accommodate turning maneuvers at Yield-controlled intersections to current AASHTO policy.

ping. Figure 39 compares the recommended values to the values used in the current AASHTO policy for ISD Case II.

Consideration of Sight Triangles for Both Crossing and Turning Maneuvers

In many cases, the length of each leg of the sight triangle for the crossing maneuver exceeds the length of the corresponding leg of the sight triangle for turning maneuvers. In this case, only the sight triangle for crossing maneuvers need be checked. However, if the length of the major-road leg of the sight triangle for the turning maneuver exceeds the length of the corresponding leg for the crossing maneuver, then both sight triangles should be checked.

Recommended Sight Distance Policy for Three-Leg Yield-Controlled Intersections

At three-leg Yield-controlled intersections, with Yield control on the stem of the T-intersection, no crossing maneuver is possible so only the sight triangle shown in Figure 35 need be checked. This generally results in much shorter recommended ISD values for three-leg Yield-controlled intersections than for four-leg Yield-controlled intersections. The recommended legs of the clear sight triangle for three-leg Yield-controlled intersections are compared to current AASHTO policy in Figures 38 and 39.

Discussion of Recommended Policy

It is evident from the policy recommendations presented above that the sight distance needs of minor-road vehicles are greater at Yield-controlled intersections, especially four-leg Yield-controlled intersections, than at Stop-controlled intersections. This results from the greater freedom of action afforded the driver approaching a Yield sign. If no potentially conflicting traffic is present, the driver approaching a Yield sign can proceed without stopping, while the driver approaching a Stop sign is required to stop whether potentially conflicting traffic is present or not. This greater freedom of action requires that the driver on a Yield-controlled approach be able to see a greater length of the major road at a greater distance from the intersection than the driver on a Stop-controlled approach. Where these greater sight distances cannot be provided economically, the simplest response is to install a Stop sign rather than a Yield sign. Designers and traffic engineers should be aware that there is a "cost" in additional sight distance to installing a Yield sign and Yield signs should not be used where that additional sight distance cannot be provided.

Incorporation of Changes to the SSD Model Recommended by Fambro et al.

Fambro et al. has recommended several changes to AASHTO SSD policy in a forthcoming NCHRP project final report. The recommendations for ISD policy for Yield-

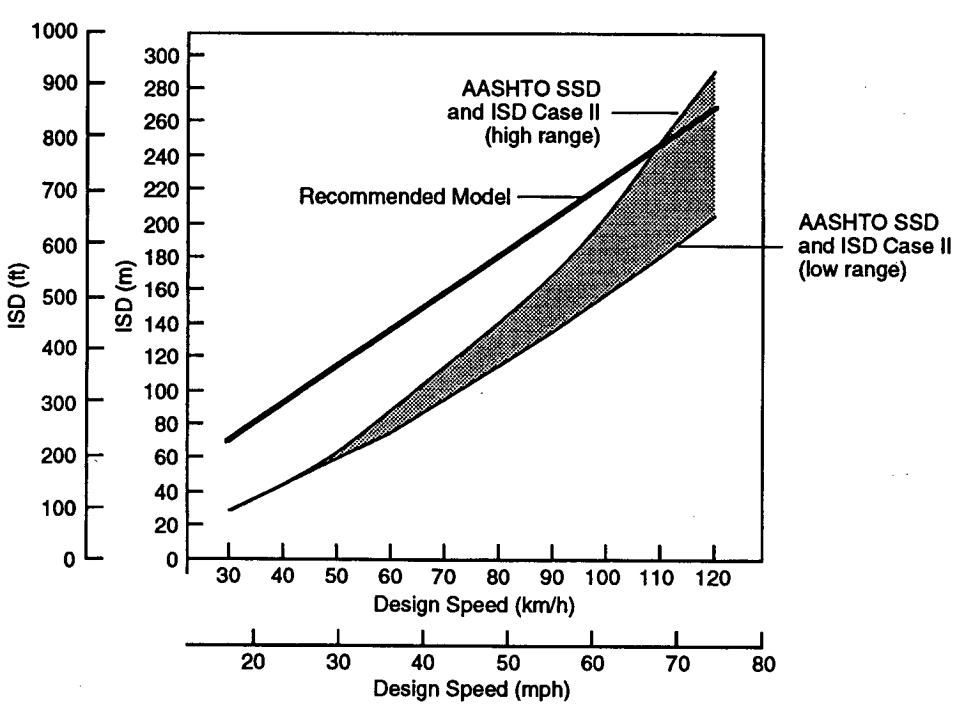


Figure 39. Comparison of recommended values for the leg of the sight triangle along the major road to accommodate turning maneuvers by passenger cars at Yield-controlled intersections to current AASHTO policy.

TABLE 33 Leg of Sight Triangle Along the Minor Road for Crossing Maneuvers from a Yield-Controlled Approach (modified for consistency with the stopping sight distance model recommended by Fambro et al. [*4*])

Design speed (km/h)	Length of minor-road leg of sight triangle (m)	Current AASHTO SSD criteria[a] (m)	Design speed (mph)	Length of minor-road leg of sight triangle (ft)	Current AASHTO SSD criteria[b] (ft)
30	30	30	20	100	125
40	40	45	25	130	150
50	55	65	30	160	200
60	65	85	35	195	250
70	80	115	40	235	325
80	100	140	45	275	400
90	115	170	50	320	475
100	135	205	55	370	550
110	155	250	60	420	650
120	180	290	65	470	725
			70	530	850

[a] High end of range in 1994 Green Book (rounded).
[b] High end of range from 1990 Green Book.

TABLE 34 Travel Time (t_a) for Minor-Road Vehicle to Reach the Major Road on a Yield-Controlled Approach (modified for consistency with the stopping sight distance model recommended by Fambro et al. [*4*])

Minor-road design speed (km/h)	Travel time for minor-road vehicle,[a] t_a (sec)	Minor-road design speed (mph)	Travel time for minor-road vehicle, t_a (sec)
30	3.6	20	3.7
40	4.0	25	4.0
50	4.4	30	4.3
60	4.8	35	4.6
70	5.1	40	4.9
80	5.5	45	5.2
90	5.9	50	5.5
100	6.3	55	5.8
110	6.7	60	6.1
120	7.0	65	6.4
		70	6.7

controlled intersections presented above are based, in part, on parameter values used in the current AASHTO SSD policy. If the recommendations made by Fambro et al. are adopted by AASHTO for use in SSD policy, corresponding changes to the ISD recommendations for Yield-controlled intersections should be considered for consistency. In particular, the recommended ISD criteria for crossing maneuvers at Yield-controlled intersections would change, but the criteria for turning maneuvers would not. Table 33 presents a replacement for Table 31, and Table 34 presents a replacement for Table 32, which should be used if the recommendations by Fambro et al. are adopted by AASHTO. As in the case of uncontrolled intersections in Chapter 2, the Fambro et al. recommendations result in slightly longer intersection sight distances at low speeds and in moderately shorter sight distances at higher speeds.

CHAPTER 5

EVALUATION OF ISD POLICY FOR INTERSECTIONS WITH TRAFFIC SIGNAL CONTROL

This chapter presents an evaluation of the ISD requirements for intersections with traffic signal control.

CURRENT AASHTO POLICY

The AASHTO policy for ISD Case IV states that sight distance based on the Case III procedures should be available at signalized intersections. Traffic signals greatly reduce the need for ISD requirements by alternating the right-of-way between the intersection approaches according to a fixed or variable cycle. However, the rationale for the AASHTO policy is that Case III should be applied at signalized intersections to assure that adequate sight distance is available for four situations:

- Signal malfunctions;
- Signal violations by motorists;
- Flashing operation of signals; and
- Right-turn-on-red by motorists.

The policy also states that, when determining the sight lines (i.e., the clear sight triangle) for the design maneuvers (i.e., crossing maneuver, left turn, or right turn), the designer should consider the effects of roadside appurtenances, parked cars, snow accumulation, or any other restriction to the sight line.

The AASHTO policy for Case IV also emphasizes the need for motorists to be able to see the signal soon enough to perform the action it indicates.

The AASHTO policy for Case IV has not changed between the 1984 and 1990 Green Books, or between the 1990 and 1994 Green Books. However, the underlying policy for ISD Cases IIIB and IIIC was changed between 1984 and 1990 (see Chapter 2 of this report).

CURRENT HIGHWAY AGENCY POLICIES

Table 35 summarizes the design policies used by state and local highway agencies for ISD at signalized intersections. Approximately 58 percent of state highway agencies use either the 1984 or 1990 AASHTO Green Book policy for ISD Case IV. However, 21 percent of state highway agencies have their own state policies for Case IV and another 21 percent of state highway agencies have no policy concerning sight distance at signalized intersections. It should be noted that the survey on which the table is based was conducted before the publication of the 1994 Green Book.

The Case IV policies of the 10 state highway agencies whose policies differ from AASHTO are as follows:

- Three states have Case IV policies based on their own policies for Cases IIIB and IIIC that differ from AASHTO.
- Two states use the AASHTO policy for Case IIIA rather than the policies for Cases IIIB and IIIC.
- One state uses its own ISD policy for *all* intersections, including signalized intersections (see Table 5).
- Two states have policies that use language similar to Case IV, but do not explicitly refer to Case IIIB. Case IIIC is mentioned only for intersections where right turn on red is permitted.
- Two states have policies that make no explicit reference to Case III at all. These policies state that sight distance must be available so that drivers are provided with "some view of the intersection."

Table 35 shows that the AASHTO policy for ISD Case IV is used by only 49 percent of local agencies in urban areas and only 26 percent of local agencies in rural areas. Local agencies are more likely than state agencies to have no formal policy for sight distance at signalized intersections.

ASSESSMENT OF CURRENT POLICIES

The AASHTO policy for ISD Case IV represents a fail-safe approach to sight-distance requirements at signalized intersections. Most engineers would have little argument that adequate ISD for Case III is needed when a signal is placed on flashing operation and that adequate ISD for Case IIIC is needed when right turn on red is permitted. However, the provision of sight distance for signal malfunctions and signal violations at intersections where neither flashing operation nor right turn on red is permitted, is based on the occurrence of very-low-probability events. This issue is addressed in the subsequent evaluation of alternative methodologies.

TABLE 35 Summary of Highway Agency Design Policies for ISD Case IV

Design policy	Number (percentage) of agencies					
	State highway agencies		Local agencies			
			Urban		Rural	
1990 AASHTO Green Book	21	(44.7)	11	(29.7)	5	(21.7)
1984 AASHTO Green Book	6	(12.8)	7	(18.9)	1	(4.3)
Own state policy	10	(21.3)	1	(2.7)	3[a]	(13.0)
Own local policy	–	–	7	(18.9)	1	(4.3)
Don't consider this case	10	(21.3)	11	(29.7)	13	(56.5)
	47		37		23	

[a] These local agencies use design policies for ISD Case IV based on the policy of the state highway agency in their state.

ALTERNATIVE ISD MODELS AND METHODOLOGIES

It appears obvious to the authors that, if ISD Case IV is retained in AASHTO policy, the sight-distance requirements at locations where Case IV is applicable should be the same as those for ISD Case III. The analyses presented above for Case III at Stop-controlled intersections are generally applicable, as well, to maneuvers at signalized intersections that are not directly controlled by the signal.

The need also appears obvious to retain the requirement that motorists be able to see the signal soon enough to perform the action it indicates. Where obstructions restrict the visibility of a signal to approaching motorists, a Signal Ahead sign may be needed.

The key issue addressed in evaluation of Case IV is to determine what operational conditions at signalized intersections require explicit ISD criteria. The following alternatives were identified for analysis:

- Require sight distance for ISD Case IIIB and IIIC for minor-road approaches if the signal is placed on flashing operation during low-volume periods;
- Require sight distance for ISD Case IIIC for approaches from which right turn on red is permitted; and
- Eliminate ISD requirements based on signal malfunctions and signal violations by motorists.

Each of these alternatives is evaluated below.

ISD Requirements for Flashing Operation

Traffic signals are often placed on flashing operation during low-volume periods (e.g., at night). Such intersections typically operate with flashing-red operation on one or more minor-road approaches and flashing-yellow operation on two or more major-road approaches. The following discussion is not applicable to operation with flashing-red indications on all approaches, but such operation—for which the ISD requirements are minimal—is not typically used for intersections that operate as normal traffic signals during the remainder of the day.

The importance of providing adequate sight distance for signalized intersections during flashing operation can be demonstrated by direct analogy to sight distance requirements for two-way Stop-controlled intersections. If the ISD policy for two-way Stop-controlled intersections is justified, then the provision of sight distances for Case III for left- and right-turn maneuvers also appears to be justified at signalized intersections with flashing operation. During the period of flashing operation, the intersection operates, in effect, with two-way Stop control and should have the same ISD requirements.

Since flashing operations are normally used only during low-volume periods, typically at night, the question might be raised as to whether these low volumes might justify reduced sight-distance criteria. However, it should be recognized that the nighttime volumes at many signalized intersections may be higher than the peak volumes at some two-way Stop-controlled intersections on local roads and streets where ISD Case III is applicable.

The only other possible mitigating factor that could possibly permit reduced sight-distance levels at signalized intersections with flashing operations is that, since flashing operation is used primarily at night, vehicle headlights on the pavement may indicate the approach of a vehicle on the major road before it can actually be seen by the minor-road driver. While this undoubtedly contributes to nighttime safety at intersections, there is no evidence to suggest that the presence of headlights would be effective in making an otherwise inadequate intersection operate safely.

Therefore, it is recommended that the provision of sight distance for left and right turns from the minor-road approaches at signalized intersections with two-way flashing red operation be retained. Highway agencies that design and operate signalized intersections without sufficient sight distance to satisfy ISD Case III for left- or right-turn maneuvers should be encouraged to have operational policies that permit two-way red-flashing operation only at intersections where the availability of adequate sight distance has been verified. Of course, if the sight-distance criteria for ISD Case III are modified as recommended in Chapter 3 of this report, this change would apply to Case IV as well as to Case III.

ISD Requirements for Right-Turn-on-Red Maneuvers

The sight-distance requirements to make a right turn on red at a signalized intersection are no different than the sight-distance requirements to make a right turn from a minor-road approach at a two-way Stop intersection. Right-turn-on-red maneuvers at signalized intersections are made at all times of day and under all traffic-volume conditions. If anything, the higher average traffic-volume levels at signalized intersec-

tions make the provision of sufficient sight distance for right turns more critical. Therefore, there appears to be no justification for not requiring adequate sight distance for Case III for right turns from signalized intersection approaches where right turn on red is permitted. Highway agencies that design and operate intersections without adequate sight distance for Case III for right turns should be encouraged to have operational policies that prohibit right turn on red on intersection approaches without adequate sight distance.

For the reasons stated above, the retention of ISD Case IV is recommended for intersection approaches where right-turn-on-red maneuvers are permitted. The recommended modifications of ISD Case III presented in Chapter 3 of this report would also be applicable to ISD Case IV.

ISD Requirements for Signal Malfunctions and Signal Violations by Motorists

Part of the AASHTO Green Book rationale for ISD Case IV is that sight distance is needed at all signalized intersections because (1) the signal may malfunction at times and go dark; and (2) motorists may intentionally or unintentionally violate the signal. Thus, clear sight triangles based on ISD Case III are currently required at signalized intersections even if neither flashing operation nor right turn on red is permitted. This appears to be a policy judgment that is not supported by any research.

No data are available on the frequency of accidents related to signal malfunctions and unintentional signal violations, but the frequency of such accidents is undoubtedly very small. Furthermore, it does not appear desirable to base sight-distance requirements for signals on the potential for intentional signal violations. Sight distance to accommodate illegal maneuvers is not provided for any other case; e.g., the potential for Stop sign violations is not considered in the sight-distance requirements for Stop-controlled intersections.

Finally, it should be recognized that, at an unsignalized intersection with limited sight distance and a history of sight-distance-related accidents, the installation of a signal (or an all-way Stop) might be considered by a highway agency as a solution to the existing safety problem that may be more cost-effective than reconstructing the intersection to increase sight distance. One of the MUTCD warrants for signal installation is the occurrence in a one-year period of five or more accidents susceptible to correction by signalization (*21*). Yet, AASHTO policy for ISD Case IV currently requires the provision of the full Case III sight distance for every signalized intersection. It appears to the authors that highway agencies should have the option to signalize an intersection as a countermeasure to sight-distance-related accidents at intersections where it is not feasible to improve the sight distance. Thus, we recommend eliminating signal malfunctions and signal violations as a consideration in ISD Case IV. However, we do recommend that flashing operation of the signals and right turn on red be permitted at such intersections only where adequate sight distance for Case III can be provided.

RECOMMENDATIONS

The following recommendations are made based on the evaluation of ISD Case IV in this research:

- The policy of providing sight distance for ISD Case III for both left and right turns at signalized intersections where flashing operation is used during low traffic-volume periods should be retained. Highway agencies should be encouraged to have traffic-control policies that make sight distance a consideration in determining whether flashing operation should be permitted.
- The policy of providing sight distance for ISD Case III for right turns from signalized intersection approaches on which right-turn-on-red maneuvers are permitted should be retained. Highway agencies should be encouraged to have traffic control policies that make sight distance a consideration in determining whether a right turn on red should be permitted on an intersection approach.
- Signal malfunctions and signal violations should be eliminated as a justification for providing Case III sight distance at signalized intersections.
- The recommended revisions to the ISD criteria for Stop-controlled intersections (Case III) also should apply to signalized intersections (Case IV). However, the Case III sight-distance requirements need not be considered at intersections where the signal will not be placed on two-way flashing-red operation and at which right turn on red will not be permitted.

CHAPTER 6

OTHER ISSUES RELATED TO INTERSECTION SIGHT DISTANCE DESIGN

This chapter discusses other issues related to intersection sight distance including left turns from the major road, intersections with all-way Stop control, ramp terminals, effect of intersection skew, effect of vertical profiles of intersection approaches, ISD measurement rules, and supplementary ISD requirements for Stop-controlled intersections.

LEFT TURNS FROM THE MAJOR ROAD

Current AASHTO Policy

A relatively new issue in intersection sight distance is the consideration of the sight-distance requirements for left turns from the major road. Clearly, motorists that wish to turn left from the major road into a minor road or driveway must yield the right of way to oncoming traffic on the major road. Turning vehicles, therefore, must wait for an appropriate gap in opposing traffic (or the change to a red signal for the opposing traffic) to complete their turn. The drivers of turning vehicles can identify appropriate gaps in opposing traffic only if adequate sight distance is available from the position of the left-turning vehicle along the opposing direction of travel on the major road.

Until 1990, there was no AASHTO policy concerning the sight-distance requirements for left turns from the major road. However, a policy on this issue was added in the 1990 Green Book and, in the 1994 Green Book, this policy was designated as ISD Case V. This policy states that:

> Required sight distance for a stopped vehicle turning left from a major highway into a minor highway (or entrance) may be computed from the formula $D = 0.28V(J + t_a)$, where V = design speed on the mainline, J = 2 sec, and t_a = the time required to accelerate and traverse the distance to clear traffic in the approaching lane. Acceleration time may be obtained directly from Figure IX-33.

The equation quoted above is formally defined as follows:

$$ISD = 0.278V(J + t_a) \quad (49)$$

where:

ISD = ISD required for a left turn off of the major road (ft)
V = speed of vehicle on major road (km/h)
J = perception-reaction time for driver of turning vehicle (sec) (assumed: J = 2 sec)
t_a = time required for the turning vehicle to travel a length equal to W + L
W = width of opposing lanes (m) which represents the length of the vehicle's turning path
L = length of the turning vehicle (m)

It should be noted that Equation (49) is the same equation currently used for ISD Case IIIA, although its interpretation is different.

The time required to complete the turn (t_a) should be based on acceleration from a stop, which is assumed to require more sight distance than a left turn by a moving vehicle that does not stop.

Figure 40 illustrates the distances involved in the AASHTO policy for left turns off the major road.

Current Highway Agency Policies

In the survey of state and local highway agencies presented in Appendix B, the only reported policies concerning sight-distance requirements for left turns from the major road were

- The Green Book policy that is quoted above; and
- Reliance on the provisions of SSD along the major road to provide adequate sight distance for left turns from the major road.

ASSESSMENT OF CURRENT POLICIES

The ISD Case V policy for left turns from the major road may be redundant in many cases. At a typical four-leg intersection with two-way Stop control, the following sight distances should be available from other current provisions of AASHTO policy:

- SSD for both directions of travel on the major road;
- ISD for Case IIIB for left turns from both minor-road approaches; and
- ISD for Case IIIC for right turns from both minor-road approaches.

Figure 41 illustrates the sight lines and clear sight triangles that are provided by these policies. It is apparent from the figure that there are very few possible locations for sight obstruc-

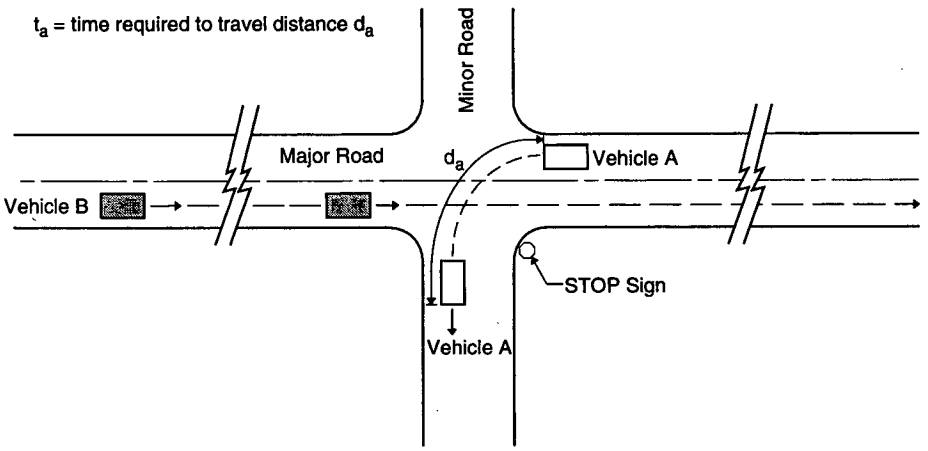
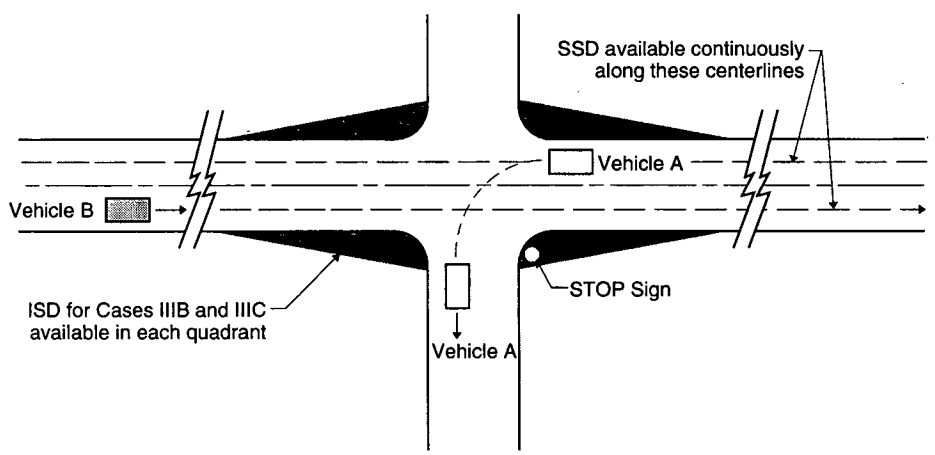
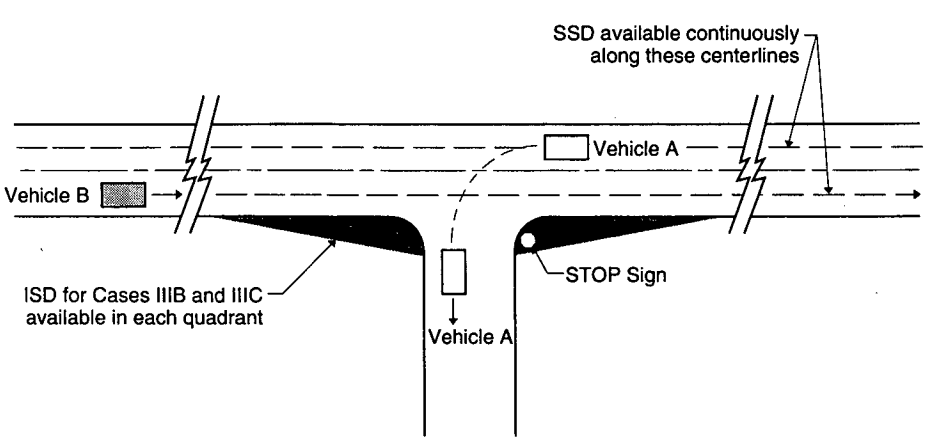


Figure 40. Distances involved in AASHTO ISD policy for left turns from the major road.



Four-Leg Intersection



Three-Leg Intersection

Figure 41. Sight lines and clear sight triangles provided at four-leg and three-leg intersections.

tions that could limit the sight distance between Vehicle A, the left-turning vehicle, and Vehicle B, the oncoming vehicle in the opposing direction of travel. In particular:

- At least on undivided roads, Vehicles A and B should have essentially the same SSD toward each other as toward the road straight ahead of them.
- Where vertical curvature is the limiting factor for SSD, the provision of SSD to a 150-mm (6-in) object should provide even greater SSD from both Vehicles A and B toward one another, since both vehicles should be approximately 1.3 m (4.25 ft) high. This ensures that if Vehicle A is turning when Vehicle B first sights Vehicle A, then Vehicle B has sufficient sight distance to stop. While this ensures safety, it is not desirable operationally since Vehicle B has the right of way.
- The availability of clear sight triangles for ISD Cases IIIB and IIIC in all four quadrants of the intersection makes it unlikely that SSD along the major road will be limited by horizontal sight obstructions. In other words, even if there is a horizontal curve on one of the major road approaches, the provisions of ISD Cases IIIB and IIIC for both minor-road approaches make it unlikely that there could be a substantial sight obstruction on the inside of that horizontal curve. Therefore, vertical curvature is more likely than horizontal curvature to control SSD.

On the other hand, some intersections may have substantial sight-distance concerns for left-turning vehicles, even where SSD and ISD Cases IIIB and IIIC are provided.

- At T-intersections, the absence of clear sight triangles for ISD Cases IIIB and IIIC normally found in two of the four quadrants of the intersection could restrict sight distance for left-turning vehicles where horizontal curves are present on the major road.
- On divided highways, despite the provision of SSD along each roadway, sight obstructions in the median could limit left-turn sight distance.
- An opposing left-turn vehicle may limit the sight distance for a vehicle turning left. Such sight-distance limitations occur only when two vehicles are turning left simultaneously.

As explained in Appendix B, highway agencies are about equally divided on whether sight-distance requirements for left turns from the major road are needed in the AASHTO Green Book (see Tables B-7 and B-19).

Research by Micsky (*22*) included field studies of left turns from the major road at two intersections in Pennsylvania. Micsky found that the acceleration times in Green Book Figure IX-33 agreed well with the 50th percentile of the observed acceleration times for left turns by stopped vehicles. For both intersections studied, the clearance distance was approximately 14 m (47 ft). Table 36 compares the field data with the current AASHTO acceleration times for the specified distance.

Micsky also tried to measure perception-reaction time for left turns in the field by defining perception-reaction time as beginning when an opposing vehicle came within a critical distance of the stopped left-turning vehicle and ending when the left-turning vehicle began to accelerate from a stop to make its turn. The critical distance was defined as the distance that the opposing major-road vehicle would travel at its observed speed in the time that was subsequently required for the left-turn vehicle to complete its turn. In other words, Micsky assumed that as soon as it became infeasible for the left-turning driver to accept one gap, the perception-reaction time for the next gap began.

Micsky observed 50th percentile perception-reaction times in the range of 2.5 to 3.0 sec, which exceeds the recommended AASHTO value of 2.0 sec. However, it must be recognized that Micsky defined perception-reaction time much differently than AASHTO. It is implicit in Equation (49) that perception-reaction time begins when the gap in opposing traffic begins (i.e., when the previous opposing vehicle passes the stopped left-turning vehicle). However, Micsky's data suggest that most of the perception-reaction time occurs *before* the opposing vehicle reaches the left-turning vehicle because the driver of the left-turning vehicle is evaluating the approaching gap. This suggests that the J term in Equation (49) is much smaller than 2.0 sec and may, in fact, be close to zero.

The sight restrictions created by opposing left-turn vehicles on divided highways can be minimized by the use of parallel and tapered offset left-turn lanes, as shown in Figure 42.

Alternate ISD Models and Methodologies

The following two alternative ISD methodologies were considered as candidates for left turns from the major road:

- An alternative model based on gap acceptance, equivalent to that recommended for turns onto the major road at Stop-controlled intersections; and
- Elimination of explicit sight-distance requirements for left turns from the major road, and reliance on SSD and ISD Cases IIIB and IIIC to ensure that adequate sight distance is available for left turns.

These alternative methodologies are evaluated in the next section.

Evaluation of Alternative ISD Models and Methodologies

An analysis of alternative ISD methodologies for left turns from the major road is presented below.

TABLE 36 Comparison of Field Data for Left Turns from Major Road with Current AASHTO Acceleration Times

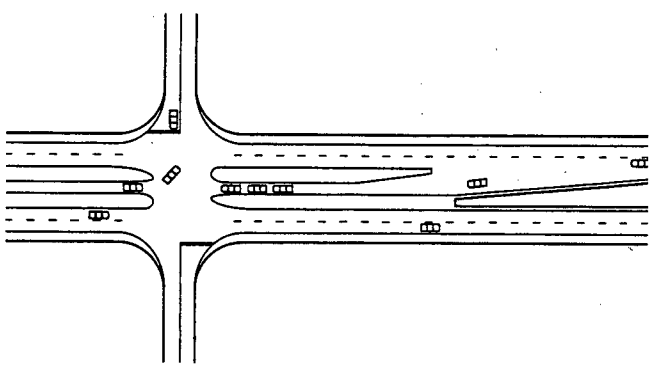
Intersection No.	Observed acceleration time (sec)[a] 50th percentile	85th percentile	Acceleration time (sec) from Green Book Figure IX-33[b]
1	3.84	4.60	4.30
2	4.24	5.05	4.30

[a] From Reference 24.
[b] Acceleration time for a distance of 14 m (47 ft).

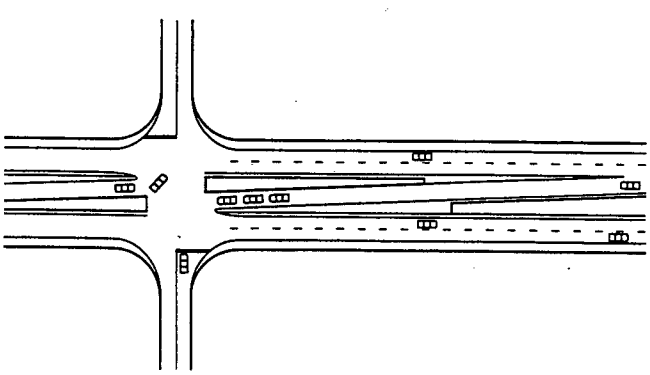
Alternative Model Based on Gap Acceptance

As in Case III, an alternative methodology based on gap acceptance can be used as an alternative to the acceleration model in Equation (49). Such a model would have the same form as the gap acceptance model in Equation (32).

The field study by Kyte et al. ("Capacity Analysis of Unsignalized Intersections," draft final report of NCHRP Project 3-46, 1995), which has been discussed in Chapter 3 of this report, has recommended a critical gap value of 4.2 sec for left turns from the major road by passenger cars for inclusion in the unsignalized intersection analysis procedures of the *Highway Capacity Manual* (*HCM*) (*14*). A constant value was recommended regardless of the number of lanes to be crossed. A heavy-vehicle adjustment of 1.0 sec for two-lane highways and 2.0 sec for multilane highways was also recommended.



Parallel offset left-turn lanes



Tapered offset left-turn lanes

Figure 42. Parallel and tapered offset left-turn lanes to minimize sight restrictions from opposing vehicles turning left.

The Micsky study (*22*) discussed above also evaluated gap-acceptance behavior for left turns from the major road at two Pennsylvania intersections. For these two intersections, the critical gaps with 50 percent probability of acceptance determined from logistic regression were 4.6 and 5.3 sec. This is consistent with the experience reported in Chapter 3 that the Kyte et al. data represent higher volume intersections where drivers appear motivated to accept shorter gaps.

As in Chapter 3, it is reasonable that design policies should be more conservative than operational criteria such as the *HCM*. Therefore, a critical gap for left turns from the major road of 5.5 sec is recommended. This value was chosen to be slightly higher than either of the observed values from the Micsky study. The recommended 5.5-sec value is substantially more conservative than the recommended *HCM* value of 4.2 sec; this is consistent with the recommended gap-acceptance values for sight distance design at Stop-controlled intersections (see Chapter 3), which are also substantially more conservative than the corresponding *HCM* values. It is recommended that the critical gap be increased to 6.5 sec for left turns by single-unit trucks and 7.5 sec for left turns by combination trucks. In addition, if the number of opposing lanes to be crossed exceeds one, add 0.5 sec per additional lane for passenger cars and 0.7 sec per additional lane for trucks, as for left turns onto the major road in Chapter 3.

Eliminate Sight-Distance Requirements for Left Turns from the Major Road

The possibility of eliminating the sight-distance requirement for left turns from the major road has been considered. Our review indicates that separate sight-distance criteria are not necessary at four-leg intersections on undivided roadways where SSD is provided along each roadway and clear sight triangles for ISD Cases IIIB and IIIC are provided in each quadrant. The other SSD and ISD criteria ensure that adequate sight distance will be available for left turns from the major road

making the left-turn sight-distance criteria redundant. For the same reason, left-turn criteria are redundant for three-leg intersections on undivided highways on tangent alignment where the other types of sight distance are provided.

However, it is recommended that AASHTO policy should include sight-distance requirements for left turns from the major road that apply to three-leg intersections on or near horizontal curves and intersections on divided roadways (i.e., roadways with medians). This can be stated in terms of a policy that applies to all intersections and driveways where left turns from the major road are permitted but only need to be checked for three-leg intersections on or near horizontal curves and for divided-highway intersections.

Recommendations

The following recommendations have been developed for left turns from the major road:

- A sight-distance policy for left turns from the major road should be presented in AASHTO policy as a separate case.
- This case should be applicable to all intersections and driveways, but need only to be checked for three-leg intersections on or near horizontal curves and intersections on divided roadways (roadways with medians). At four-leg intersections on undivided roadways and three-leg intersections on tangent undivided roadways, provision of SSD on each roadway and provision of ISD Cases IIIB and IIIC in each quadrant will ensure adequate sight distance for left turns from the major roadway.
- Sight-distance policy for left turns from the major road should be based on a gap-acceptance approach for compatibility with the recommended sight-distance policy for left and right turns from the minor road at Stop-controlled intersections. The critical gaps that should be used to determine sight distance for left turns from the major road are presented in Table 37.
- At intersections on divided highways, the use of parallel and tapered offset left-turn lanes should be considered to minimize the sight restrictions created by opposing left-turn vehicles, as shown in Figure 42.

TABLE 37 Recommended Travel Times for Determining Sight Distance for Left Turns from the Major Road Across Opposing Traffic Lanes

Vehicle type	Travel time (sec) at design speed of major road
Passenger car	5.5
Single-unit truck	6.5
Combination truck	7.5

Note: For left turns that must cross more than one opposing lane, add 0.5 sec per additional lane for passenger cars and 0.7 sec per additional lane for trucks.

INTERSECTIONS WITH ALL-WAY STOP CONTROL

Prior to the 1994 edition, the Green Book recommended that the criteria for ISD Case III be applied to all-way Stop intersections. This recommendation has been removed from the 1994 edition because there is no apparent reason at an intersection with all-way Stop control to require as much sight distance as at a two-way Stop intersection. An intersection with all-way Stop control can operate safely as long as a driver at the stop line on any of the approaches has a view of the first vehicle stopped at the stop line on each of the other approaches. Additional sight distance is desirable, but not necessary. Indeed, it may be very appropriate to convert an existing two-way Stop-controlled intersection with a history of sight-distance-related accidents to all-way Stop control to reduce these accidents if improvements to increase the available sight distance are infeasible.

RAMP TERMINALS

Prior to the 1994 edition, the Green Book contained a sight-distance procedure for ramp terminals. This procedure was developed prior to the AASHTO policies for ISD Cases IIIB and IIIC and has been made redundant by those policies. Therefore, it is recommended that the Green Book should not contain a separate sight-distance policy for ramp terminals. For sight-distance design purposes, ramp terminals should be treated like any other Stop-controlled or signal-controlled intersection. However, designers should be very conscious of potential sight obstructions such as bridge railings, piers, and abutments that are likely to be found near ramp terminals.

EFFECT OF INTERSECTION SKEW

The 1994 Green Book includes a section on the effect of oblique-angle or skewed intersections on ISD policies. Each element of this current policy and its applicability to the revised ISD policies recommended in this report is presented below.

- The Green Book states that the policy on effect of skew applies to highways that intersect at an angle less than 60 degrees and for which realignment to increase the angle of the intersection is not justified. This policy appears appropriate and it is recommended that it be retained.
- The Green Book illustrates the shape of the clear sight triangles that result at an oblique angle intersection, as illus-

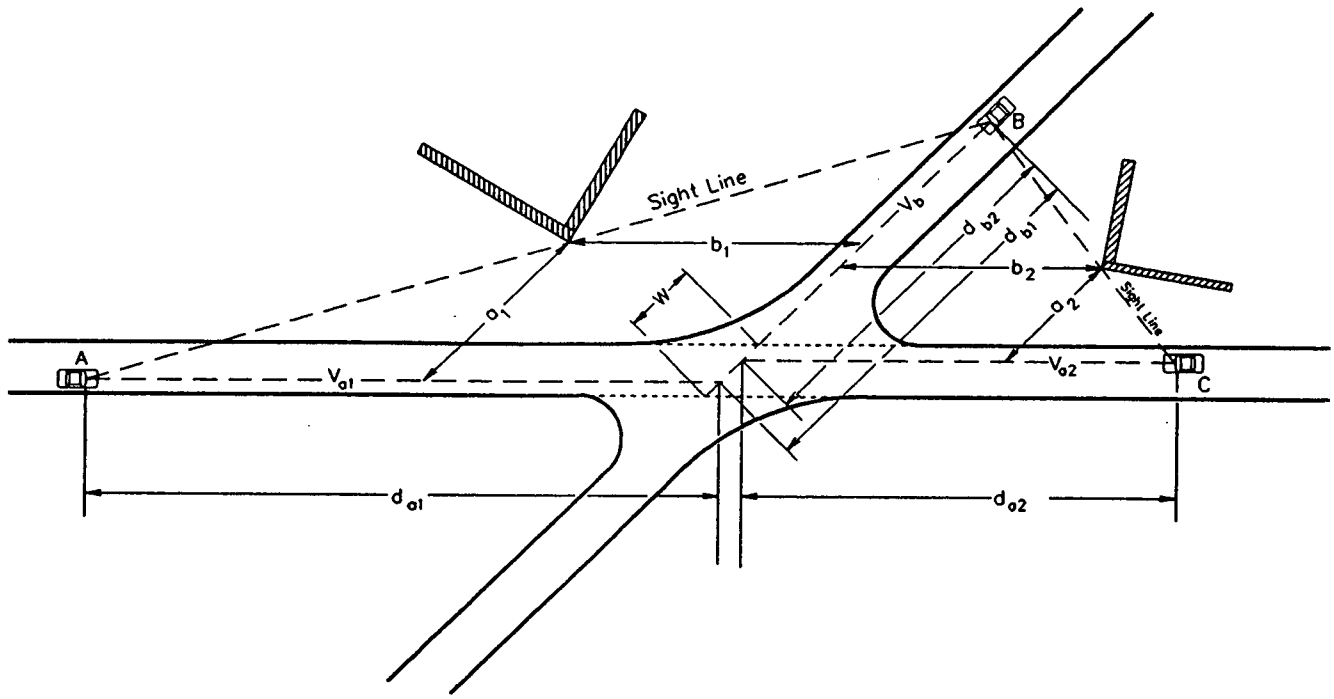


Figure 43. Clear sight triangles at a skewed intersection (1).

trated in Figure 43, which is based on Green Book Figure IX-43. The policy states that the legs of the sight triangle should lie along the approach legs of the intersection, even when the legs do not intersect at a right angle. This results in clear sight triangles that are larger or smaller than those at right-angle intersections. This policy remains appropriate and it is recommended that it be retained.

- The Green Book notes that, in the obtuse-angle quadrant of an oblique-angle intersection, the angle between the approach leg and the sight line is small so that drivers can look across the full sight triangle with only a small head movement. However, the policy notes that in the acute-angle quadrant, drivers would be required to turn their heads considerably to see across the entire clear sight triangle. For this reason, the Green Book recommends that ISD Case I not be applied to oblique-angle intersections and that the criteria for Cases II and III, whichever is larger, be applied instead. It is recommended that this policy be retained.
- The Green Book states that the path across an oblique-angle intersection is longer than across a right-angle intersection and that therefore the value of W in Equation (21) should be increased by dividing the actual traveled-way width by the sine of the intersection angle. The gap-acceptance approach recommended in this report for Stop requires a slightly modified approach to this issue. The recommended approach is to compute the actual path length in a turning or crossing maneuver as the total width of the lanes to be crossed divided by the sine of the intersection angle. If the actual path length exceeds the total width of the lanes to be crossed by 3.6 m (12 ft) or more, then use the actual path length to determine an appropriate number of additional lanes to consider in applying the multilane adjustment shown in Tables 26 and 27.

EFFECT OF THE VERTICAL PROFILES OF INTERSECTION APPROACHES

All of the ISD criteria in the Green Book and all of the recommended sight distances in this report are based on the assumption that each of the intersecting roadways has a level grade. However, some adjustment to the sight-distance criteria is appropriate when upgrades or downgrades are present. Two types of adjustments are appropriate: one adjustment for approach sight triangles and the other for departure sight triangles.

Each of the approach sight triangles recommended in this report is based on a variation of the AASHTO SSD model. The Green Book states that grades up to 3 percent have little effect on SSD. For grades above 3 percent, the Green Book includes a grade term in the computation of SSD as follows:

$$SSD = 0.278Vt_{pr} + \frac{V^2}{254(f \pm G)} \qquad (50)$$

where:

SSD = stopping sight distance (m)
V = roadway design speed (km/h)

t_{pr} = perception-reaction time (sec) (assumed t_{pr} = 2.5 sec)
f = coefficient of braking friction (for assumed values see Green Book Table III-1)
G = percent grade divided by 100 (+ for upgrades, − for downgrades)

Table 38 shows the ratio of SSD for a level roadway and SSD for upgrades and downgrades above 3 percent. This table is appropriate to adjust the dimensions of approach sight triangles, as explained below.

For approach grades up to 3 percent, no adjustment of intersection sight distances based on grade is recommended. As in the current Green Book, it is recommended that intersection approach grades up to 6 percent be permitted if the intersection sight distances are adjusted.

For an uncontrolled intersection approach at which the approach grade exceeds 3 percent, the leg of the clear sight triangle along that approach should be increased or decreased by the appropriate ratio shown in Table 35.

For a Yield-controlled intersection for which the minor-road grade exceeds 3 percent, the legs of the clear sight triangle for the crossing maneuvers along both the minor and major roads should be increased or decreased by the ratios shown in Table 38. It should be noted that the adjustment to both the minor-road and major-road legs of the sight triangle are based on the grade on the minor-road approach.

The departure sight triangles recommended in this report are based on the concept of gap acceptance. Appropriate grade adjustments are based on the field study results of Kyte et al., as shown in Table 18. For Stop-controlled intersections for which the minor road has an upgrade that exceeds 3 percent, the travel times (based on gap acceptance) used to compute the leg of the clear sight triangle along the major road should be increased as follows:

- Right turn from minor road +0.1 sec/% grade
- Left turn from minor road +0.2 sec/% grade
- Crossing maneuver from minor road +0.2 sec/% grade

For example, if the minor-road approach to a Stop-controlled intersection has a 4 percent upgrade, then the travel-time value used to compute the major-road leg of the clear sight triangle would be increased from 7.5 to 7.9 sec for right turns by passenger cars and from 7.5 to 8.3 sec for left turns by passenger cars, and from 6.5 to 7.3 sec for crossing maneuvers by passenger cars. These same adjustments also apply to departure sight triangles at Yield-controlled and signal-controlled intersections to which the policy for Stop-controlled intersections is applicable. No adjustment for the grade of the minor-road approach is needed unless the rear wheels of the design vehicle would be on an upgrade exceeding 3 percent when the vehicle is stopped on the approach; thus, the need for an adjustment for the grade of the minor-road approach can be avoided by providing a relatively level roadway section, with grades less than or equal to 3 percent on the minor-road approach, for at least the length of a selected design vehicle.

ISD MEASUREMENT RULES

Three key criteria used in the measurement of ISD in the field are driver eye height, vehicle height, and vehicle stopping position on Stop-controlled approaches. These criteria are used in determining whether objects located within a clear sight triangle are, in fact, sight obstructions. Each of these criteria is discussed below.

Driver Eye Height

All ISD cases in current AASHTO policy are based on a driver eye height of 1,070 mm (3.50 ft) as are the ISD design policies of all but one state highway agency. Appendix C shows that other countries use driver eye heights in the range from 1,000 to 1,150 mm (3.28 to 3.77 ft). Thus, it appears that U.S. policy is in the center of the range of current international practice. Fambro et al. conducted field studies of driver eye height in a forthcoming NCHRP project ("Determination of Stopping Sight Distances," final report) and obtained the results shown in Table 39. Based on these results, Fambro et al. has recommended a value of 1,080 mm (3.54 ft) for driver eye height in SSD design and, therefore, we recommend use of the same value for ISD design. This recommended 1,080-mm (3.54-ft) value for driver eye height was found by Fambro

TABLE 38 Adjustment Factors for Approach Sight Distance Based on Approach Grade

Approach grade (%)	Design speed (km/h)									
	30	40	50	60	70	80	90	100	110	120
-6	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2
-5	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2
-4	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
-3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+5	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+6	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

Note: Based on ratio of stopping sight distance on specified approach grade to stopping sight distance on level terrain.

TABLE 39 Field Study Results for Distribution of Driver Eye Heights

Descriptive statistic	Driver eye height, mm (ft)		
	Passenger cars	Multipurpose vehicles[a]	Heavy trucks
Sample size	875	629	163
Mean	1,149 (3.77)	1,482 (4.86)	2,447 (8.03)
Standard deviation	55 (0.18)	130 (0.43)	107 (0.35)
Maximum value	1,422 (4.67)	2,034 (6.67)	2,816 (9.24)
Minimum value	955 (3.13)	1,053 (3.45)	2,103 (6.90)
5th percentile	1,060 (3.48)	1,264 (4.15)	2,304 (7.56)
10th percentile	1,082 (3.55)	1,306 (4.28)	2,329 (7.64)
15th percentile	1,094 (3.59)	1,331 (4.37)	2,341 (7.68)

[a] Includes pickup trucks, minivans, vans, and sport/utility vehicles.

et al. to represent the 10th percentile of the passenger car driver population.

Several sight-distance policies based on the sight-distance needs of heavy trucks are recommended in this report. Where the sight-distance value used in design is based on a single-unit or combination truck as the design vehicle, it also is appropriate to use the eye height of a truck driver in the design of vertical curvature. Fambro et al. indicates that the 10th percentile driver eye height for a heavy truck is 2,330 mm (7.64 ft), and this value is recommended for use in design of vertical curves to accommodate ISD. In such cases, the vertical-curve lengths appropriate for both passenger cars and trucks should be checked and the longer vertical curve should be used. It is also appropriate in such cases to check whether particular objects within the clear sight triangle are sight obstructions to passenger car drivers, truck drivers, or both.

Object Height

The AASHTO object height of 1,300 mm (4.25 ft) is used for ISD by all but one state highway agency in the United States; that state (Connecticut) uses an object height of 1,070 mm (3.50 ft). The object heights used by other countries in ISD design are generally lower than those used in the U.S. For example, France and Germany use object heights of 1,000 mm (3.28 ft), while Australia uses 1,150 mm (3.77 ft). In all three countries, the object height used in ISD design is the same as the driver eye height.

In ISD design, the object which the driver must be able to see is a vehicle on another approach to the intersection. The object height used by AASHTO for ISD design generally is understood to represent the height of a passenger car. Table 40 shows the results of a recent field study of vehicle heights by Fambro et al. performed as part of a forthcoming NCHRP project. The table shows that the 15th percentile of the current distribution of passenger car heights is slightly higher than the current AASHTO design value of 1,300 mm (4.25 ft) for vehicle height; for both multipurpose vehicles (pickup trucks, minivans, vans, and sport/utility vehicles) and heavy trucks, the 15th percentile of the vehicle-height distribution is substantially greater than the current AASHTO design value.

TABLE 40 Field Study Results for Distribution of Vehicle Heights

Descriptive statistic	Vehicle height, mm (ft)		
	Passenger cars	Multipurpose vehicles[a]	Heavy trucks
Sample size	1,378	987	158
Mean	1,384 (4.54)	1,759 (5.77)	3,590 (11.78)
Standard deviation	59 (0.19)	155 (0.51)	581 (1.91)
Maximum value	1,690 (5.54)	2,501 (8.21)	4,639 (15.22)
Minimum value	1,156 (3.79)	1,279 (4.20)	2,396 (7.86)
5th percentile	1,282 (4.21)	1,523 (5.00)	2,652 (8.71)
10th percentile	1,315 (4.31)	1,564 (5.13)	2,719 (8.92)
15th percentile	1,331 (4.37)	1,613 (5.29)	2,714 (9.10)

[a] Includes pickup trucks, minivans, vans, and sport/utility vehicles.

The current AASHTO policy does not address how much of the vehicle needs to be seen for it to be recognized as a potentially conflicting vehicle. It seems likely that a potentially conflicting vehicle would not be recognized as such if only its roofline were visible. There are no available guidelines for how much of a vehicle needs to be seen for it to be recognized. However, the likelihood that drivers will detect and recognize other vehicles is enhanced because a driver stopped at or approaching an intersection is actively looking for potentially conflicting vehicles. Given this alerted state, a driver looking for a potentially conflicting vehicle is likely to treat any moving object detected as such, at least until the object gets closer and can be recognized.

Object detection is affected by many variables including driver visual acuity, the contrast between the object and its background, and the ambient light level. However, assuming 20/40 driver visual acuity and optimal conditions (including high contrast between the vehicle and its background), it should be possible for the driver to recognize a portion of the vehicle subtending 2 min of arc. To allow for nonoptimum conditions, we recommend that this value be doubled to 4 min of arc.

For Stop-controlled intersections, the recommended sight distances along the major road for a passenger car (shown in Figure 29) range from 65 m (210 ft) at a major-road design speed of 30 km/h (18 mph) to 250 m (820 ft) at a major-road design speed of 120 km/h (75 mph). Using the 4-min-of-arc criterion described above, the amount of the vehicle that needs to be seen would range from 80 mm (0.26 ft) for a 30-km/h (18 mph) approach to 290 mm (0.95 ft) for a 120-km/h (75-mph) approach. These data indicate that by subtracting the amount of the vehicle that needs to be seen from the 15th percentile passenger-car height, the object height appropriate for use in design should range from 1,250 mm (4.10 ft) at a design speed of 30 km/h (18 mph) to 1,040 mm (3.41 ft) at a design speed of 120 km/h (75 mph). While it would be justifiable to let the object height vary with design speed because the apparent size of the object to be seen varies with distance, it is recommended that the practice of using a single object height be retained.

The lower end of the range of object heights determined above, 1,040 mm (3.41 ft), is very close to 1,070 mm (3.50 ft), which is the value of driver eye height currently used in ISD design and also very close to 1,080 mm (3.54 ft), the revised value of driver eye height recommended by Fambro et al. Therefore, the use of an object height of 1,080 mm (3.54 ft), equivalent to driver eye height, is recommended for use in ISD design. This incorporates into ISD policy the concept that, if one driver can "look the other driver in the eye," then that driver can see enough of the other driver's vehicle to recognize it as a vehicle. This also makes ISD reciprocal; if one driver can see the other vehicle, then that vehicle's driver can also see the first vehicle. Each of the other countries where ISD policies on this issue were available—Australia, France, and Germany—use an object height equal to driver eye height. It should be recognized that the recommended object height is more conservative than necessary for roadways with lower design speeds.

Multipurpose vehicles have become very prevalent and now comprise over one-third of the combined passenger car/multipurpose vehicle fleet. Consideration was given to determining the 15th percentile vehicle height based on a combination of the distributions of passenger car and multipurpose vehicle height shown in Table 40. However, it was found that this would only raise the recommended object height by about 10 mm (0.03 ft). Because this difference is so small, it was decided not to incorporate consideration of multipurpose vehicles in the determination of object height. Therefore, recommendations are based on consideration of the passenger-car fleet alone. This makes the recommended object height very conservative because the recommended criteria should not only permit 85 percent of passenger cars to be seen, but also every multipurpose vehicle and heavy truck as well.

The choice of the object height used for ISD may affect both the identification of objects that should be removed or lowered within the clear sight triangle and the design of vertical curves on the intersecting roadway. However, with the recommended object height of 1,080 mm (3.54 ft), the vertical-curve lengths along the major road required by the recommended ISD criteria for Stop-controlled intersections are less than the vertical-curve lengths required by the current AASHTO SSD criteria, except for vertical curves with algebraic differences in grade of 6 percent or more and design speeds less than 60 km/h (37 mi/h). Even in these cases, the additional vertical-curve length does not exceed 15 m (50 ft).

Vehicle Stopping Position

Current AASHTO policy assumes a distance of 6 m (20 ft) from the edge of the major road to the driver's eye for stopped vehicles at Stop-controlled intersections. This distance is used by all but eight state highway agencies. The state policies for the distance from the edge of the major road to the driver's eye that differ from AASHTO are as follows:

- One state uses 5.2 m (17 ft).
- Three states use 4.6 m (15 ft).
- One state uses 4.3 m (14 ft).
- One state uses a minimum value of 3 m (10 ft), and a desirable value of 6 m (20 ft).
- One state assumes that the driver will stop the front of his or her vehicle at the edge of the shoulder or curb offset. The driver's eye is assumed to be 3 m (10 ft) behind the front of the vehicle.
- One state assumes that the driver will stop with the front of hisor her vehicle at the edge of the shoulder or the marked stop bar. If there is no shoulder or stop bar, it is assumed that the front of the vehicle will be 1.2 m (4 ft)

from the edge of the major road for public road intersections [or 0.6 m (2 ft) for driveways]. The driver's eye is assumed to be 2.1 m (7 ft) behind the front of the vehicle. This can result in a distance from the edge of the major road to the driver's eye of 3.4 m (11 ft) for intersections [or 2.7 m (9 ft) for driveways].

The 6-m (20-ft) distance from the edge of the major road to the driver's eye used in the current AASHTO policy is based on the sum of two distances: 3 m (10 ft) from the edge of the major-road traveled way to the front of the stopped vehicle and 3 m (10 ft) from the front of the vehicle to the driver's eye. Each of these components was examined in field studies reported in Appendix H.

The field study of vehicle stopping positions reported in Appendix H found that, where necessary to obtain a clear view, minor-road drivers will stop with the front of their vehicle substantially closer than 3 m (10 ft) to the major-road traveled way. Vehicle stopping positions on nine intersection approaches were evaluated, measuring from the edge of the major-road traveled way to the front of the stopped vehicle for the final stopping position from which the vehicle accelerated to enter or cross the major road. On the two approaches where drivers stopped closest to the major-road traveled way, the mean stopping positions were 0.9 and 1.4 m (2.8 and 4.7 ft) and the 85th percentile stopping positions were 1.5 and 2.0 m (5.0 and 6.6 ft). Based on these data, a vehicle stopping position of 2.0 m (6.6 ft) from the edge of the major-road traveled way to the front of the stopped vehicle is recommended as a basis for design. Drivers often stop farther from the major road than this, but the field data show that they can comfortably stop with the front of their vehicle within 2.0 m (6.6 ft) of the major road, when necessary. This is a conservative choice because, on the two intersection approaches where it was necessary to stop close to the major road, at least 85 percent of the vehicles observed stopped closer to the major road than the recommended design value.

On the other hand, it is recognized that more generous design is desirable. Therefore, it is recommended that the distance from the edge of the major-road traveled way to the front of the stopped vehicle should be at least 2.0 m (6.6 ft) and, where feasible, 3.0 m (10.0 ft).

A field study was also conducted to determine a design value for the distance from the front of the vehicle to the position of the driver's eye. As explained in Appendix H, this was accomplished by measuring a sample of 101 passenger cars, pickup trucks, and minivans observed in a parking lot. The distribution of the driver's eye position, relative to the front of the vehicle, was as follows:

Minimum	1.8 m (5.8 ft)
Mean	2.2 m (7.3 ft)
Maximum	2.9 m (9.5 ft)
Median	2.2 m (7.3 ft)
85th percentile	2.4 m (8.0 ft)

Based on these data, an appropriate design value is 2.4 m (8.0 ft), which represents both the 85th percentile and the 90th percentile of the observed distribution.

When the recommended design values of 2.0 m (6.6 ft) for the distance from the edge of the major-road traveled way to the front of the stopped vehicle and 2.4 m (8.0 ft) from the front of the vehicle to the drivers eye are combined, the resulting distance from the edge of the major road to the driver's eye is 4.4 m (14.4 ft). This value is recommended as the leg of the departure sight triangle along the major road for Stop-controlled intersections. Where feasible, it is desirable to increase this distance from 4.4 m (14.4 ft) to 5.4 m (17.7 ft).

SUPPLEMENTARY ISD POLICIES FOR STOP-CONTROLLED INTERSECTIONS

Two state highway agencies and many local agencies have adopted policies that encourage or require supplementary ISD at Stop-controlled intersections. There is no equivalent guideline in AASHTO policy.

The Mississippi State Highway Department, in addition to its other ISD criteria for Cases I through IV, requires a sight triangle known as a "sight flare" with legs along the major- and minor-roads equal to the low-range values of the AASHTO SSD policy. This sight flare is required at all intersections of a major road that have a design hour volume (DHV) greater than 300 veh/hr with a minor road that has a current average daily traffic (ADT) greater than 300 veh/day. Most such intersections have Stop-control or signal control and the Mississippi policy is equivalent to providing Case II ISD at such intersections, when only Case III ISD normally would be provided.

The Mississippi sight flare policy is a design requirement, not just a guideline. However, the policy recognizes that it may be impractical at existing intersections to remove existing obstructions within the sight triangle, especially in urban areas.

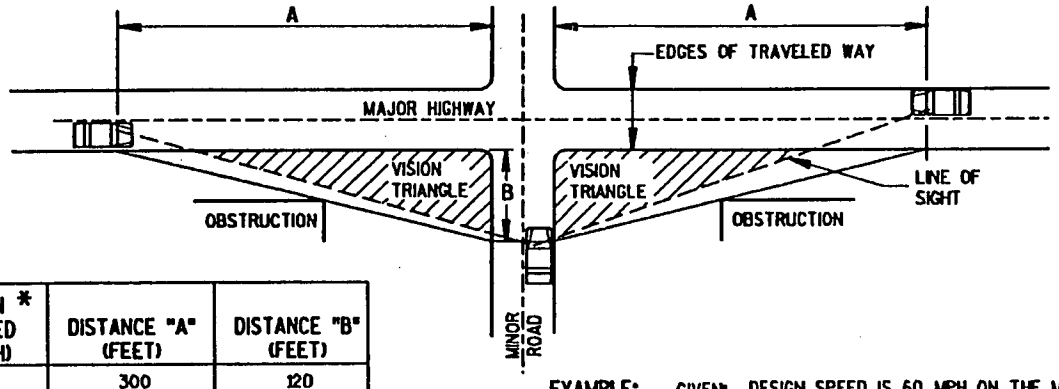
The Wisconsin Department of Transportation has a similar policy on supplementary sight triangles at Stop-controlled intersections. However, this policy, which is illustrated in Figure 44, represents desirable sight distances and is not a design requirement. This policy encourages the use of a clear sight triangle that extends substantially farther along the minor road than the 6 m (20 ft) required by ISD Case III. The leg of the sight triangle along the major road is within the range between the lower and higher values of SSD required by AASHTO policy. The Wisconsin policy states explicitly that its purpose is to accommodate potential Stop-sign violations by the minor-road vehicles. This Wisconsin policy applies only to removal of sight obstructions in the clear sight triangle, and is not intended to require changes in horizontal or vertical curvature to provide the desirable ISD.

Many local agencies have policies that require clear sight triangles at all intersections, including Stop-controlled and signal-controlled intersections. Such policies are typically

implemented though local ordinances that prohibit sight obstructions on private property within the specified sight triangle. Sight triangles of this type typically extend 9 to 21 m (30 to 70 ft) along the minor road, so they require a larger leg of the sight triangle along the minor road than is required by AASHTO policy for ISD Case III.

Supplementary ISD policies of the type discussed above have the potential to improve safety at Stop-controlled intersections. However, because they are intended to accommodate the sight-distance requirements of an illegal maneuver (a Stop-sign violation), such policies should be considered desirable guidelines, not design requirements.

GUIDE DIMENSIONS FOR VISION TRIANGLES STOP CONTROL ON MINOR ROAD



DESIGN * SPEED (MPH)	DISTANCE "A" (FEET)	DISTANCE "B" (FEET)
40	300	120
50	400	150
55	500	160
60	600	175

* USE THE DESIGN SPEED OF THE MINOR ROAD TO DETERMINE DISTANCE "B".

EXAMPLE: GIVEN: DESIGN SPEED IS 60 MPH ON THE MAJOR HIGHWAY AND 50 MPH ON THE MINOR HIGHWAY.

SOLUTION:

READING FROM THE DIMENSION TABLE, THE LEGS OF THE VISION TRIANGLE ARE "A" = 600 FEET AND "B" = 150 FEET.

NOTES:
1. DISTANCES ARE APPROXIMATE AND MAY BE ADJUSTED TO FIT SITE CONDITIONS. THE INTENT IS TO PROVIDE THE OPERATOR OF A VEHICLE ON THE MAJOR HIGHWAY AN EXTRA FIELD OF VIEW AND TIME TO ALTER THE VEHICLES SPEED AS NECESSARY IN THE EVENT A VEHICLE APPROACHING THE INTERSECTION ON THE MINOR ROAD FAILS TO STOP.
2. THESE GUIDELINES ARE FOR THE VISION TRIANGLE ONLY AND SHOULD NOT BE INTERPRETED AS SATISFYING SIGHT DISTANCE REQUIREMENTS AT THE INTERSECTION DUE TO VERTICAL OR HORIZONTAL CURVES.
3. THE VISION TRIANGLE MUST BE FREE OF ALL OBSTRUCTIONS INCLUDING ANY CUT SLOPES.

Figure 44. Supplementary ISD criteria for Stop-controlled intersections used in Wisconsin.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

This chapter summarizes the conclusions and recommendations of the research.

CONCLUSIONS

The following conclusions concerning intersection operations relevant to ISD were reached during the research:

1. Drivers approaching uncontrolled intersections on urban and suburban residential streets typically slow to about 50 percent of their midblock running speed, even if no potentially conflicting vehicles are present on an intersecting approach.
2. Drivers turning left and right from Stop-controlled approaches typically accept gaps in major-road traffic that are much shorter than the sight distances currently specified for ISD Cases IIIB and IIIC of the AASHTO Green Book. The observed gap-acceptance behavior in some cases requires deceleration by major-road vehicles at modest, comfortable rates to speeds that are typically about 70 percent of their upstream running speed. All of the intersections at which these measurements were made had good safety records.
3. The average acceleration rates used by passenger cars turning left or right onto a major road correspond closely to the design values for passenger cars used by AASHTO in Figure IX-33 and Table IX-8 of the 1994 Green Book.
4. Neither the model currently used by AASHTO for left and right turns from Stop-controlled approaches, nor the modification of it that was developed during the research, can be adequately calibrated without field data on conflicts between turning vehicles accelerating onto the major road and following major-road vehicles that are much more critical or severe than the maneuvers observed in the field during the research. The current AASHTO model would require data for emergency or near-emergency conditions to be properly calibrated. The very conservative assumptions currently used by AASHTO in this model result in sight distances that are much longer than necessary, based on the gap-acceptance data that were collected in the field during the research (see Conclusion 2).
5. Drivers approaching Yield-controlled intersections on urban and suburban residential streets typically slow to about 60 percent of their midblock running speed, even if no potentially conflicting vehicles are present on an intersecting approach.
6. The current AASHTO policy, by recommending the same sight distances at signalized intersections as are used at Stop-controlled intersections, could be interpreted as discouraging signalization as a cost-effective alternative to reconstruction for existing intersections that have experienced (or new intersections that might experience) accidents related to limited sight distance. A less-restrictive policy should be considered, while recognizing that at intersections where flashing signal operations or right turn on red are permitted, the sight distance needs are similar to Stop-controlled intersections.
7. Where available sight distance on a Stop-controlled approach is limited and where drivers on the minor road can obtain a better view of major-road traffic by moving closer to the major road, drivers on the minor road will move their vehicles closer to the major road than the 3-m (10-ft) distance currently used in AASHTO policy. At two intersections with limited sight distance studied in the field, the 85th percentile distance from the edge of the major-road traveled way to the front of the stopped vehicle was 2.0 m (6.6 ft).
8. In a sample of parked vehicles whose dimensions were measured, every vehicle had a shorter distance from the front of the vehicle to the estimated position of the driver's eye than the 3-m (10-ft) value currently used in AASHTO policy. Both the 85th and 90th percentiles of the measured distance from the front of the vehicle to the estimated position of the driver's eye was 2.4 m (8.0 ft).

RECOMMENDATIONS

The recommendations of the study for each aspect of ISD-design policy are presented below. Recommended revisions to the text of the AASHTO Green Book that implement these recommendations are presented in Appendix J.

General

1. A new approach to ISD-design policy should be implemented to introduce a more consistent concep-

tual basis for ISD models and to set revised values for ISD design based on those models.

2. The approach sight triangles used at uncontrolled and Yield-controlled intersections should provide approaching vehicles sufficient sight distance to stop, if necessary, before reaching the intersection. The dimensions of the approach sight triangles should be based on the same model used for SSD design, with appropriate modifications to incorporate the actual speed profiles that approaching drivers are observed to use in the field at intersection with various types of traffic control.
3. The departure sight triangles used at locations where stopped vehicles must enter or cross a major road should be based on the concept of gap acceptance. The leg of the departure sight triangle along the major road should be based on the gap-acceptance behavior of drivers observed in the field. The leg of the departure sight triangle along the minor road should be based on vehicle stopping positions observed in the field.
4. Each approach or departure sight triangle should be reviewed as part of the intersection-design process to determine whether either the terrain or specific roadside objects located within the clear sight triangle constitute sight obstructions. Any sight obstructions should be removed or lowered where possible.

Intersections with No Control

5. Approach sight triangles at intersections with no control (other than the right-of-way rule) should be based on a variation of the SSD model. The perception-reaction time for this case should be 2.5 sec, rather than 2.0 sec used in current policy. The speed profile for vehicles approaching the intersection should be based on field observations that show that approaching vehicles typically slow to 50 percent of their mid-block running speed, even if no potentially conflicting vehicles are present on the intersecting approach.
6. The recommended ISD values for intersections with no control are presented in Table 10. If the recommendations of Fambro et al. ("Determination of Stopping Sight Distances," the final report of a forthcoming NCHRP project) concerning stopping sight distance are adopted by AASHTO, then the recommended ISD values in Table 11 should be used in place of those in Table 10.
7. Future research may be needed to provide better data for determining sight distance needs for uncontrolled intersections under higher-speed rural conditions. However, the success of such research may be limited by very low traffic volumes that exist at most intersections of this type.

Intersections with Stop Control on the Minor Road

8. The leg of the departure sight triangle along the major road for both left and right turns from a Stop-controlled approach should be based on gap-acceptance values (or critical gaps) of 7.5 sec for passenger cars, 9.5 sec for single-unit trucks, and 11.5 sec for combination trucks. Appropriate adjustments should be made to these values for the number of lanes on the major road and for the approach grade on the minor road. The length of the sight triangle along the major road should be the distance that would be traversed in the critical-gap time by a vehicle traveling at the design speed of the major road. The appropriate design vehicle should be selected based on the anticipated traffic mix on the minor road.
9. The leg of the departure sight triangle along the minor road for both left and right turns should generally have a length of 4.4 m (14.4 ft) from the edge of the major-road traveled way to the anticipated position of the driver's eye in a vehicle on the minor road. This recommended value is based on the sum of 85th percentile stopping position of 2.0 m (6.6 ft) on intersection approaches with limited sight distance discussed in Conclusion 7, and the 85th and 90th percentile distance of 2.4 m (8.0 ft) from the front of a vehicle to the driver's eye discussed in Conclusion 8. Where feasible, it is desirable to increase this distance from 4.4 m (14.4 ft) to 5.4 m (17.7 ft).
10. The sight distance needed to cross a major road from a stopped position is generally less than the sight distance need to turn left or right onto the major road unless the width of the major road exceeds six lanes. Recommended critical gaps for crossing the major road are presented in Table 27. Appropriate adjustments should be made to these values for the number of lanes on the major road and the approach grade on the minor road.

Intersections with Yield Control on the Minor Road

11. At four-leg Yield-controlled intersections, two types of approach sight triangles should be considered, one based on the sight distance needed for the crossing maneuver and one based on the sight distance needed for left- and right-turn maneuvers. At three-leg Yield-controlled intersections, no crossing maneuver is feasible so only the approach sight triangle for left- and right-turn maneuvers needs to be considered.
12. For the crossing maneuver, the recommended lengths of the leg of the approach sight triangles along the minor road, both to the left and to the right, are shown in Table 31. These lengths are based on a modification of the current AASHTO SSD model based on the field observa-

tion that vehicles on Yield-controlled approaches typically slow to 60 percent of their midblock running speed, even if no potentially conflicting traffic is present on the intersecting approach. The values shown in Table 31 should be adjusted, as appropriate, for the grade of the Yield-controlled approach. The leg of each sight triangle along the major road should be based on the distance that would be traversed by a vehicle traveling at the design speed of the major road in the time shown in Table 32, plus a crossing and clearance interval, as indicated in Equation (48).

13. For left- and right-turn maneuvers, the recommended length of each approach sight triangle along the minor road is 25 m (80 ft). The length of the sight triangles along the major road, both to the left and to the right, should be based on the critical gaps given in Table 26 for Stop-controlled intersections, with appropriate adjustments for the number of lanes on the major road and the minor-road approach grade, plus 0.5 sec. This 0.5-sec increase in travel time represents the net effect of greater deceleration time, but shorter acceleration time, for a turning vehicle on a Yield-controlled approach in comparison to a vehicle on a Stop-controlled approach.
14. Each Yield-controlled approach should have departure sight triangles, both to the left and to the right, like those provided for Stop-controlled approach to accommodate the sight-distance needs of vehicles that are forced to stop by the presence of conflicting traffic on the major road.

Intersections with Traffic Signal Control

15. Intersections with traffic-signal control generally need sufficient sight distance for the driver of the first vehicle on each approach to see the first vehicle stopped on each of the other approaches.
16. For traffic signals that will be placed on flashing operation during low-volume periods, with a flashing red indication facing the minor-road approaches and a flashing yellow indication facing the major-road approaches, the departure sight triangles recommended for left and right turns from Stop-controlled approaches (and, where appropriate, crossing maneuvers from Stop-controlled approaches) should be provided.
17. For signalized approaches from which right turn on red will be permitted, the departure sight triangles for right turns from a Stop-controlled approach should be provided.

Left Turns from the Major Road

18. At locations where left turns from the major road are permitted at intersections and driveways, at unsignalized intersections and at signalized intersections without a protected left-turn phase, sight distance along the major road should be provided based on the critical gaps shown in Table 37. Appropriate adjustments should be made for the number of lanes to be crossed by the left-turning vehicle. Normally, the provision of SSD along the major road and ISD for left and right turns from the minor road will ensure that sufficient sight distance for left turns from the major road is available. However, the availability of sight distance for left turns from the major road should be reviewed explicitly for three-leg intersections on or near horizontal curves and for divided-highway intersections.
19. The use of parallel and tapered offset left-turn lanes, like those shown in Figure 42, should be considered at divided-highway intersections to minimize the sight-distance restrictions that result from opposing vehicles turning left.

ISD Measurement Rules

20. Measurements to determine whether specific objects within the sight triangle are sight obstructions should be based on a driver eye height of 1,080 mm (3.54 ft) and an object height of 1,080 mm (3.54 ft). These values also should be used in the design of vertical curves to accommodate ISD.

Tort Liability Issues

21. As explained in Appendix I, implementation of the recommendations made in this report is not expected to have major tort liability implications for highway agencies.

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APPENDIXES A–I

Appendixes A through I as submitted by the research agency are not published herein but are available for loan on request to the NCHRP.

APPENDIX A Questionnaires for Survey of Highway Agencies

APPENDIX B Summary of Questionnaire Responses from State and Local Highway Agencies

APPENDIX C Intersection Sight Distance Policies of Highway Agencies in Other Countries

APPENDIX D Probability of Vehicle-Vehicle Conflicts at Uncontrolled Intersections

APPENDIX E Alternative ISD Models Considered in the Research but Not Recommended

APPENDIX F Field Studies of Driver Gap-Acceptance Behavior and Acceleration/Deceleration Rates in Turning Maneuvers at Stop-Controlled Intersections

APPENDIX G Field Studies of Driver Speed Selection on Approaches to Uncontrolled and Yield-Controlled Intersections

APPENDIX H Field Studies of Vehicle Dimensions and Vehicle-Stopping Positions on Minor-Road Approaches to Stop-Controlled Intersections

APPENDIX I Tort Liability Issues Related to Intersection Sight Distance

APPENDIX J

RECOMMENDED REVISIONS TO THE AASHTO GREEN BOOK

This appendix presents recommended revisions to the AASHTO Green Book to implement the recommendations developed in this research. These recommendations will be considered by the AASHTO Task Force on Geometric Design for possible incorporation in a future edition of the Green Book.

The revised text for the Green Book presented below is intended as a complete replacement for the material concerning intersection sight distance that appears on pages 696 through 721 of the 1994 Green Book (*1*). Some of the introductory text that is still applicable has been retained, but the recommended changes are extensive enough that a complete rewriting, rather than just editing, of the existing text appeared to be appropriate.

The revised policy is based on two simple concepts: a modified stopping sight distance model for approach sight triangles and a gap-acceptance model for departure sight triangles. These concepts have been adapted to each intersection sight distance case, as appropriate. We hope that readers will find these concepts simple and easy to understand. The basic assumptions concerning each case have been included in the Green Book text, with references to this research report in appropriate places for those interested in the full derivation and supporting data for each model.

In preparing the revised text, we have found it desirable to present the ISD cases in a different order than they are presented in the current Green Book. The order of the cases was changed because the recommended sight distances for Yield-controlled intersections (currently referred to as Case II) depend, in part, on the recommended sight distance models for both uncontrolled intersections (currently referred to as Case I) and Stop-controlled intersections (currently referred to as Case III). It could be potentially confusing if an ISD case known until now as Case III were renamed as Case II. Therefore, to avoid confusing readers, we have replaced the roman numeral case numbers with letters; the ISD cases have been designated as Cases A through F, in the order in which they are presented in the revised text.

The draft text for the Green Book generally avoids recommending the type of traffic control appropriate for any particular intersection. However, the text does explain how to determine the dimensions of the clear sight triangles appropriate for any particular type of intersection traffic control.

The recommended revision to the intersection sight-distance design policy of the AASHTO Green Book is presented on the following pages.

SIGHT DISTANCE

General Considerations

Each intersection has the potential for several different types of vehicle-vehicle conflicts. The possibility of these conflicts actually occurring can be greatly reduced through the provision of proper sight distances and appropriate traffic controls. The avoidance of accidents and the efficiency of traffic operations still depend on the judgment, capabilities, and response of each individual driver.

The driver of a vehicle approaching an at-grade intersection should have an unobstructed view of the entire intersection, including any intersection traffic-control devices, and sufficient lengths of the intersecting highway to permit the driver to anticipate and avoid potential collisions. The sight distance that should be used for design under various assumptions of physical conditions and driver behavior is directly related to vehicle speeds and to the resultant distances traversed during perception-reaction time and braking.

Stopping sight distance is provided continuously along each highway or street so that drivers have a view of the roadway ahead that is sufficient to allow drivers to stop, if necessary, under prescribed conditions. The provision of stopping sight distance at all locations along each highway or street, including intersection approaches, is fundamental to safe intersection operations.

Vehicles are assigned the right of way at intersections by traffic-control devices or, where no traffic-control devices are present, by the rules of the road. A basic rule of the road is that, at an intersection at which no traffic-control devices are present, the vehicle on left must yield the right of way to the vehicle on the right if they arrive at approximately the same time. Sight distance is provided at intersections to allow the drivers of vehicles without the right of way to perceive the presence of potentially conflicting vehicles, in sufficient time for the vehicle without the right of way to stop, if necessary, before reaching the intersection. The methods for determining the sight distances needed by drivers approaching intersections are based on the same principles as stopping sight distance, but incorporate modified assumptions based on observed driver behavior at intersections.

Sight distance is also provided at intersections to allow the drivers of vehicles stopped on intersection approaches a sufficient view of the intersecting highway to decide when to turn onto the intersecting highway or to cross it. If the available sight distance for an entering or crossing vehicle is at least equal to the appropriate stopping sight distance for the major road, then drivers should have sufficient sight distance to anticipate and avoid collisions. However, in some cases, this may require a major-road vehicle to stop or slow to accommodate a turning maneuver by a minor-road vehicle. To achieve better traffic operations, so that major-road vehicles do not need to stop or slow substantially to accommodate entering or crossing vehicles, intersection sight distances that exceed stopping sight distance are desirable along the major road. Thus, intersection sight distances that exceed stopping sight distance are intended to enhance traffic operations, but are not minimum design criteria which are essential to safety.

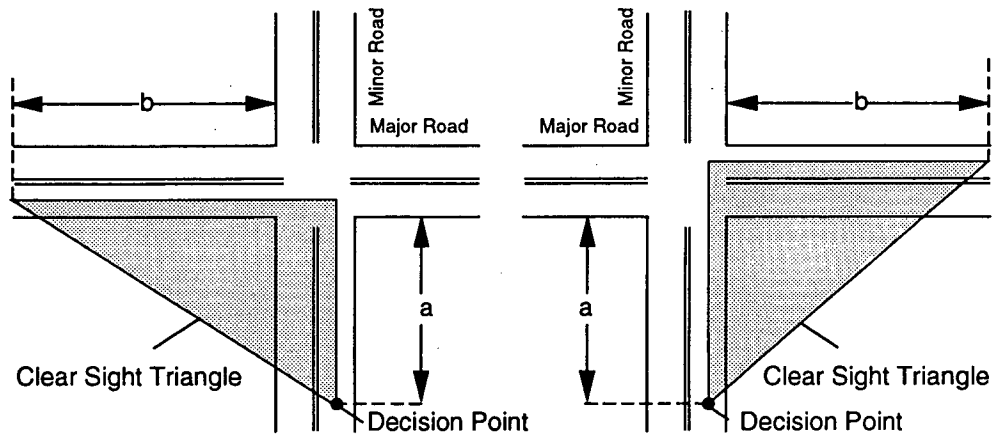
Clear Sight Triangles

Specified areas along intersection approach legs and across their included corners should be clear of obstructions that might block a driver's view of potentially conflicting vehicles. These specified areas are known as clear sight triangles. Two types of clear sight triangles considered in intersection design, approach sight triangles and departure sight triangles, are explained below. The dimensions of the clear sight triangles depend on the design speeds of the intersecting roadways and the type of traffic control used at the intersection. These dimensions are based on field studies that have observed driver behavior and have documented the space-time profiles and speed choices of drivers on intersection approaches [reference this report].

Approach Sight Triangles

Each quadrant of an uncontrolled or Yield-controlled intersection should contain a clear sight triangle free of obstructions that might block an approaching driver's view of potentially conflicting vehicles on the intersecting approaches. The area clear of sight obstructions must include sufficient lengths of both intersecting roadways, as well their included corner, so that the drivers without the right of way can see any potentially conflicting vehicle in sufficient time to slow or stop before reaching the intersection. Figure IX-32A shows typical clear sight triangles to the left and to the right for a vehicle approaching an intersection.

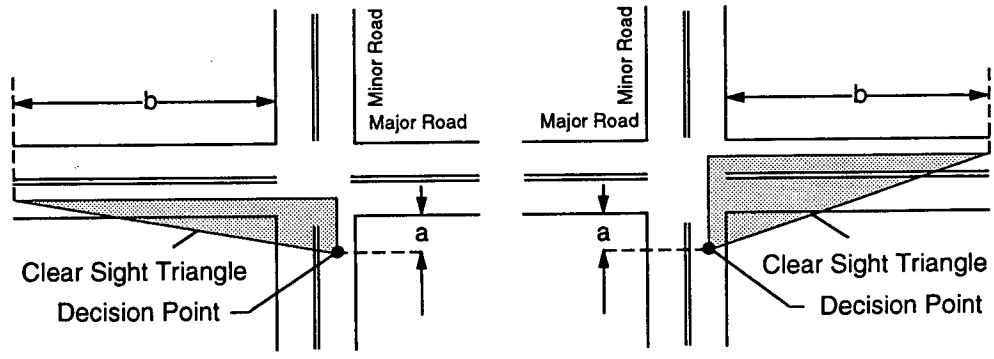
The vertex of the sight triangle on a minor-road approach (or an uncontrolled approach) represents a decision point for the minor-road driver. This decision point is the location at



Approach sight triangle for viewing traffic approaching from the left

Approach sight triangle for viewing traffic approaching from the right

A—Approach Sight Triangles



Departure sight triangle for viewing traffic approaching from the left

Departure sight triangle for viewing traffic approaching from the right

B—Departure Sight Triangles

Figure IX-32. Clear sight triangles for intersection approaches.

which the minor-road driver should begin to brake to a stop if another vehicle is present on an intersecting approach.

The geometry of a clear sight triangle is such that when the driver of a vehicle without the right of way sees a potentially conflicting vehicle on an intersecting approach that has the right of way, then the driver of that potentially conflicting vehicle can also see the first vehicle. Thus, the provision of a clear sight triangle for vehicles without the right of way also permits the drivers of vehicles with the right of way to be prepared to slow, stop, or avoid other vehicles, should it become necessary.

Approach sight triangles like those shown in Figure IX-32A are not needed for intersection approaches controlled by Stop signs or traffic signals, because the need for approaching vehicles to stop at the intersection is determined by the traffic control devices and not by the presence or absence of vehicles on the intersecting approaches.

Departure Sight Triangles

A second type of clear sight triangle provides sight distance sufficient for a driver stopped on a minor-road approach to depart from the intersection by entering or crossing the major road. Figure IX-32B shows typical departure sight triangles to the left and to the right. Departure sight triangles should be provided in each quadrant of each intersection approach controlled by Stop or Yield signs from which stopped vehicles may enter or cross a major road on which traffic is not required to stop. Departure sight triangles should also be provided for some signalized intersection approaches (see below).

The recommended dimensions of the clear sight triangle for desirable traffic operations where stopped vehicles enter or cross a major road are based on assumptions derived from field observations of driver gap-acceptance behavior on intersection approaches [reference this report]. The provision of clear sight triangles like those shown in Figure IX-32B also allows the drivers of vehicles on the major road to see any vehicles stopped on the minor-road approach and to be prepared to slow or stop, if necessary.

Identification of Sight Obstructions Within Clear Sight Triangles

The profiles of the intersecting roadways should be designed to provide the recommended sight distances for drivers on the intersection approaches. Within a clear sight triangle, any object at a height above the elevation of the adjacent roadways that would obstruct the driver's view should be removed or lowered, if feasible. Such objects may include: buildings, parked vehicles, highway structures, roadside hardware, hedges, trees, bushes, unmowed grass, tall crops, and the terrain itself. Particular attention should be given to the evaluation of clear sight triangles at interchange ramp terminals where features such as bridge railings, piers, and abutments are potential sight obstructions.

The determination of whether an object constitutes a sight obstruction should consider both the horizontal and vertical alignment of both intersecting roadways, as well as the height and position of the object. In making this determination, it should be assumed that the driver's eye is 1,070 mm above the roadway surface and that the object to be seen is 1,070 mm above the surface of the intersecting road. This object height is based on a vehicle height of 1,330 mm, which represents the 15th percentile of vehicle heights in the current passenger car population less an allowance of 260 mm, which represents a near-maximum value for the portion of the vehicle height that needs to be visible for another driver to recognize a vehicle as such. The use of an object

height equal to the driver eye height makes intersection sight distances reciprocal; i.e., if one driver can see another vehicle, then the driver of that vehicle can also see the first vehicle.

Where the sight-distance value used in design is based on a single-unit or combination truck as the design vehicle, it is also appropriate to use the eye height of a truck driver in checking sight obstructions and determining vertical curve lengths. The recommended value of truck driver eye height is 2,330 mm above the roadway surface. In such cases vertical curve lengths appropriate for both passenger cars and trucks should be checked and the longer vertical curve should be used.

INTERSECTION CONTROL

The recommended dimensions of the clear sight triangles vary with the type of traffic control used at an intersection because different types of control impose different legal constraints on drivers and, therefore, result in different driver behavior. Sight-distance policies for intersections with the following types of traffic control are presented below:

1. Intersections with no control (Case A)
2. Intersections with Stop control on the minor road (Case B)
   - Left-turn from the minor road (Case B1)
   - Right-turn from the minor road (Case B2)
   - Crossing maneuver from then minor road (Case B3)
3. Intersections with Yield control on the minor road (Case C)
   - Crossing maneuver from the minor road (Case C1)
   - Left or right turn from the minor road (Case C2)
4. Intersections with traffic signal control (Case D)
5. Intersections with all-way Stop control (Case E)

A sight-distance policy for stopped vehicles turning left from a major road (Case F) is also presented.

Intersections with No Control (Case A)

For intersections not controlled by Yield signs, Stop signs, or traffic signals, the driver of a vehicle approaching the intersection must be able to see potentially conflicting vehicles on intersecting approaches in sufficient time for the approaching driver to safely stop before reaching the intersection. The location of the vertex of the sight triangles on each approach is determined from a model that is analogous to the stopping sight distance model, with slightly different assumptions. Drivers of approaching vehicles may require up to 2.5 sec to perceive vehicles on intersecting approaches and to initiate braking.

While some perceptual tasks at intersections may require substantially less time, the detection and recognition of a vehicle that is a substantial distance away on an intersecting approach, and is near the limits of the driver's peripheral vision, may require up to 2.5 sec. The distance to brake to a stop can be determined from the same braking coefficients used for stopping sight distance in Table III-1.

Field observations indicate that vehicles approaching uncontrolled intersections typically slow down from their midblock running speed to approximately 50 percent of their midblock running speed. This occurs even when no potentially conflicting vehicles are present [reference this report]. This initial slowing typically occurs at deceleration rates up to 1.5 m/sec^2; deceleration at this gradual rate has been observed to begin even before a potentially conflicting vehicle comes into view. Braking at greater deceleration rates, which can approach those assumed in stopping sight distance, begins up to 2.5 sec after a vehicle on the intersecting approach comes into view. Thus, approaching vehicles may be traveling at less than their midblock running speed during all or part of the perception-reaction time and can, therefore, where necessary, brake to a stop from a speed less than the midblock running speed.

Table IX-7 shows the distance traveled by an approaching vehicle during perception-reaction and braking time as a function of the design speed of the roadway on which the intersection approach is located. These distances should be used as the legs of the sight triangles shown in Figure IX-32A. Referring to Figure IX-32A, highway A with an 80 km/h design speed and highway B with a 50 km/h design speed require a clear sight triangle with legs extending at least 80 m and 40 m along highways A and B, respectively.

This clear sight triangle will permit the vehicles on either road to stop, if necessary, before reaching the intersection. If the design speed of any approach is not known, it can be estimated by using the 85th percentile of the midblock running speeds for that approach.

The distances shown in Table IX-7 are generally less than the corresponding values of stopping sight distance for the same design speed. Where a clear sight triangle whose legs correspond to the stopping sight distances of their respective approaches can be provided, this will provide an even greater margin of safety. However, since field observations show that motorists slow down to some extent on approaches to uncontrolled intersections, the provision of a clear sight triangle with legs equal to the full stopping sight distance is not essential.

Where the grade along an intersection approach exceeds 3 percent, the leg of the clear sight triangle along that approach should be adjusted by multiplying by the appropriate sight distance from Table IX-7 by the appropriate adjustment factor from Table IX-8.

If the sight distances given in Table IX-7, as adjusted for grades, cannot be provided, consideration should be given to installing advisory speed signing to reduce speeds or installing Stop signs on one or more approaches.

No departure sight triangle like that shown in Figure IX-32B is required at an uncontrolled intersection because such intersections typically have very low traffic volumes.

Table IX-7. Recommended Sight Distances for Intersections with No Traffic Control (Case A).

Design speed (km/h)	Sight distance (m)
20	20
30	25
40	30
50	40
60	50
70	65
80	80
90	95
100	120
110	140
120	165

Note: For approach grades greater than 3%, multiply the sight distance values in this table by the appropriate adjustment factor from Table IX-8.

Table IX-8. Adjustment Factors for Approach Sight Distance Based on Approach Grade.

Approach grade (%)	Design speed (km/h)									
	30	40	50	60	70	80	90	100	110	120
–6	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2
–5	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2
–4	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
–3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+5	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+6	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9

Note: Based on ratio of stopping sight distance on specified approach grade to stopping sight distance on level terrain.

If a motorist finds it necessary to stop at an uncontrolled intersection because of the presence of a conflicting vehicle on an intersecting approach, it is very unlikely still another potentially conflicting vehicle will be encountered as the first vehicle departs the intersection.

Intersections with Stop Control on the Minor Road (Case B)

Departure sight triangles for intersections with Stop control on the minor road should be considered for three situations:

- Left turns from the minor road (Case B1);
- Right turns from the minor road (Case B2); and
- Crossing the major road from a minor-road approach (Case B3).

No approach sight triangles like those shown in Figure IX-32A are needed at Stop-controlled intersections because all minor-road vehicles are required to stop before entering or crossing the major road.

Left Turn from the Minor Road (Case B1)

A departure sight triangle for traffic approaching from the right like that shown in Figure IX-32B should be provided for left turns from the minor road onto the major road for all Stop-controlled approaches where left turns maneuvers are permitted. The length of the leg of the departure sight triangle along the major road should be equal to the distance traveled at the design speed of the minor road in the appropriate time shown in Table IX-9.

Field observations of the gaps in major road traffic actually accepted vehicles turning onto the major road have shown that the values in Table IX-9 provide sufficient time for the minor-road vehicle to accelerate from a stop and complete a left turn onto the major road without unduly interfering with major-road traffic operations. Field observations have also shown that major-road drivers will reduce their speed to some extent when minor-road vehicles turn onto the major road. Where the gap acceptance values in Table IX-9 are used to determine the length of the leg of the departure sight triangle along the major road, most major-road drivers should not need to reduce speed to less than 70 percent of their initial speed [reference this report].

In applying Table IX-9, it can usually be assumed that the minor-road vehicle is a passenger car. However, for minor-road approaches from which substantial volumes of heavy vehicles enter the major road, the use of the tabulated values for single-unit trucks or com-

Table IX-9. Travel Times Used to Determine the Leg of the Departure Sight Triangle along the Major Road for Left and Right Turns from Stop-Controlled Approaches (Cases B1 and B2).

Design vehicle	Travel time (sec) at design speed of major road
Passenger car	7.5
Single-unit truck	9.5
Combination truck	11.5

Adjustment for multilane highways:

For left turns onto two-way highways with more than two lanes, add 0.5 sec for passenger cars or 0.7 sec for trucks for each additional lane, in excess of one, to be crossed by the turning vehicle.

For right turns, no adjustment is necessary.

Adjustment for approach grades:

If the approach grade on the minor road is an upgrade that exceeds 3 percent:

Add 0.1 sec per percent grade for right turns

Add 0.2 sec per percent grade for left turns

bination trucks should be considered. Table IX-9 includes appropriate adjustments to the gap times for the number of lanes on the major road and for the approach grade of the minor road. The adjustment for the grade of the minor-road approach need be made only if the rear wheels of the design vehicle would be on an upgrade that exceeds 3 percent when the vehicle is at the stop line of the minor-road approach. The length of the sight triangle along the major road (distance b in Figure IX-32B) would be determined as:

$$b = 0.278V_{major}t_c$$

where:

b = length of the leg of sight triangle along the major road (m)
V_{major} = design speed of major road (km/h)
t_c = critical gap for entering the major road (sec)
(use the appropriate value from Table IX-9, as adjusted)

For example, a passenger car turning left onto the major road should be provided sight distance equivalent to a gap of 7.5 sec in major-road traffic. If the design speed of the major road is 90 km/h, this corresponds to a sight distance of 0.278(90)(7.5) or 190 m. A passenger car turning left onto a four-lane roadway will need to cross two near lanes, rather than one, which would increase the recommended gap in major road traffic from 7.5 to 8.0 sec. The corresponding value of sight distance for a 90-km/h design speed is 200 m. If the minor-road approach to such an intersection is located on a 4 percent upgrade, then the time gap selected for sight-distance design for left turns should be increased from 8.0 to 8.8 sec, equivalent to an increase of 0.2 sec for each percent grade. No adjustment of the recommended sight distance values for the major-road grade is generally required because both the major- and minor-road vehicle will be on the same grade when departing from the intersection. However, if the minor-road design vehicle is a heavy truck and the intersection is located near a sag vertical curve with grades over 3 percent, then an adjustment to extend the recommended sight distance based on the major-road grade should be considered.

If sight distances along the major road based on Table IX-9, including the appropriate adjustments, cannot be provided, consideration should be given to installing advisory speed signing on the major-road approaches.

The vertex of the departure sight triangle on the minor road should be 4.4 m from the edge of the major-road traveled way (i.e., dimension a in Figure IX-32B should be 4.4 m).

This represents the typical position of the minor-road driver's eye when a vehicle is stopped relatively close to the major road. Field observations of vehicle stopping positions found that, where necessary, drivers will stop with the front of their vehicle 2.0 m or less from the edge of the major-road traveled way. Measurements of passenger cars indicate that the distance from the front of the vehicle to the driver's eye for the current U.S. passenger car population is nearly always 2.4 m or less [reference this report]. Where feasible, it is desirable to increase the distance from the edge of the major-road traveled way to the vertex of the clear sight triangle from 4.4 m to 5.4 m, which allows 3.0 m from the edge of the major-road traveled way to the front of the stopped vehicle.

Sight distance design for left turns at divided highway intersections may require consideration of multiple design vehicles. If the design vehicle used to determine sight distance for the intersection is a passenger car and the divided highway median is wide enough to store a passenger car with adequate clearance to the through lanes at both ends of the vehicle, then no departure sight triangle for left turns is required on the minor-road approach. In most cases, the departure sight triangle for right turns should provide sufficient sight distance for a passenger car to cross near roadway of the divided highway to reach the median; possible exceptions are addressed below in the discussion of Case B3. However, a departure sight triangle should be provided for left turns by passenger cars stopped in the median. Where the median is not wide enough to store a passenger car, a departure sight triangle for left turns from the minor-road approach should be provided.

If the design vehicle used to determine sight distance for a divided highway intersection is larger than a passenger car, then sight distance for left turns may need to be checked for that selected design vehicle and for smaller design vehicles as well. If the design vehicle can be stored in the median with adequate clearance to the through lanes, then a departure sight triangle for left turns should be provided for that design vehicle turning left from the median roadway. If the design vehicle cannot be stored in the median, then a departure sight triangle should be provided for that design vehicle to turn left from the minor-road approach; furthermore, a departure sight triangle for left turns from the median roadway should be provided for the largest design vehicle that can be stored on the median roadway with adequate clearance to the through lanes. For example, if a divided highway intersection has a 12-m median width and the design vehicle for sight distance is a 21-m combination truck, then a departure sight triangle should be provided for the combination truck to turn left from the minor-road approach and a departure sight triangle should also be provided for a 9-m single unit truck to turn left from a stopped position in the median.

Right Turn from the Minor Road (Case B2)

A departure sight triangle for traffic approaching from the left like that shown in Figure IX-32B should be provided for right turns from the minor road into the major road for all Stop-controlled approaches where right turns are permitted. The lengths of the legs of the departure sight triangle for right turns should generally be the same as those for left turns used in Case B1. Specifically, the length of the leg of the departure sight triangle along the major road should be based on the travel times in Table IX-9, including appropriate adjustment factors. The length of the leg of the clear sight triangle along the minor road should be 4.4 m.

Where sight distances along the major road based on the travel times from Table IX-9 cannot be provided, it should be kept in mind that field observations indicate that, in making right turns, drivers generally accept gaps that are slightly shorter than those accepted in making left turns. The travel times in Table IX-9 can be decreased by 1.0 to 1.5 sec for right-turn maneuvers, where necessary, without undue interference with major-road traffic. While the recommended sight distance for a right-turn maneuver cannot be provided, even with a reduction of 1.0 to 1.5 sec from the values in Table IX-9, consideration should be given to installing advisory speed signing in the major-road approaches.

As indicated in Table IX-9, the adjustment for upgrades that exceed 3 percent on the minor-road approach is 0.1 sec for each percent grade.

Crossing Maneuver from the Minor Road (Case B3)

In most cases, it can be assumed that the departure sight triangles for left and right turns onto the major road, as described for Cases B1 and B2, will also provide more than adequate sight distance for minor-road vehicles to cross the major road. However, in the following situations, it is advisable to check the availability of sight distance for crossing maneuvers:

- where left and/or right turns are not permitted from a particular approach and the crossing maneuver is the only legal maneuver;
- where the crossing vehicle must cross more than six lanes; or
- where substantial volumes of heavy vehicles cross the highway and steep grades that might slow the vehicle while its rear is still in the intersection are present on the departure roadway on the far side of the intersection.

Table IX-10 presents travel times, and appropriate adjustment factors, that can be used to determine the length of the leg of the sight triangle along the major road to accommodate crossing maneuvers. At divided highway intersections, depending on the relative magnitudes of the median width and the length of the design vehicle, sight distance may need to be considered for crossing both roadways of the divided highway or for crossing the near lanes only and stopping in the median before proceeding.

Intersections with Control on the Minor Road (Case C)

If no conflicting traffic is present, drivers approaching Yield signs are permitted to enter or cross the major road without stopping. The sight distances needed by drivers on Yield-controlled approaches exceed those for Stop-controlled approaches.

For four-leg intersections with Yield control on the minor road, two separate pairs of approach sight triangles like those shown in Figure IX-32A should be provided. One set of approach sight triangles is needed to accommodate crossing the major road and a separate set of sight triangles is needed to accommodate left and right turns onto the major road. Both sets of sight triangles should be checked for potential sight obstructions.

For three-leg intersections with Yield control on the minor road, only the approach sight triangles to accommodate left- and right-turn maneuvers need be considered, because the crossing maneuver is infeasible.

Table IX-10. Travel Times Used to Determine the Leg of the Departure Sight Triangle along the Major Road to Accommodate Crossing Maneuvers at Stop-Controlled Intersections (Case B3).

Design vehicle	Travel time (sec) at design speed of major road
Passenger car	6.5
Single-unit truck	8.5
Combination truck	10.5

Adjustment for multilane highways:

For crossing a major road with more than two lanes, add 0.5 sec for passenger cars and 0.7 sec for trucks for each additional lane to be crossed.

Adjustment for approach grades:

If the approach grade of the minor road is an upgrade that exceeds 3 percent, add 0.2 sec per percent grade.

Table IX-11. Leg of Approach Sight Triangle along the Minor Road to Accommodate Crossing Maneuvers from Yield-Controlled Approaches (Case C1).

Design speed (km/h)	Distance along minor road[a] (m)	Travel time from decision point to major road (t_a)[a,b] (sec)
30	30	3.4
40	40	3.7
50	50	4.1
60	65	4.7
70	85	5.3
80	110	6.1
90	140	6.8
100	165	7.3
110	190	7.8
120	230	8.6

[a] For minor-road approach grades that exceed 3 percent, multiply by the appropriate adjustment factor from Table IX-8.
[b] Travel time applies to a vehicle that slows before crossing the intersection but does not stop.

Crossing Maneuvers (Case C1)

The length of the leg of the approach sight triangle along the minor road to accommodate the crossing maneuver from a Yield-controlled approach (distance a in Figure IX-32A) is given in Table IX-11. The distances in Table IX-11 are based on the same assumptions as those for Case A except that, based on field observations, minor-road vehicles that do not stop are assumed to decelerate to 60 percent of the minor-road design speed, rather than 50 percent. The distances and times in Table IX-11 should be adjusted for the grade of the minor-road approach using the factors in Table IX-8.

The length of the leg of the approach sight triangle along the major road to accommodate the crossing maneuver (distance b in Figure IX-32A) should be computed with the following equations:

$$t_c = t_a + \frac{w + L_a}{0.167V_{minor}}$$

$$b = 0.278V_{major}t_c$$

where:

t_c = travel time to reach and clear the major road in a crossing maneuver (sec)
b = length of leg of sight triangle along the major road (m)
t_a = travel time to reach the major road from the decision point for a vehicle that does not stop (sec) (use appropriate value for the minor-road design speed from Table IX-11, adjusted for approach grade, where appropriate)
w = width of intersection to be crossed (m)
L_a = length of design vehicle (m)
V_{minor} = design speed of minor road (km/h)
V_{major} = design speed of major road (km/h)

These equations provide a sufficient travel time for the major road vehicle during which the minor-road vehicle can: (1) travel from the decision point to the intersection, while decelerating at the rate of 1.5 m/sec^2 to 60 percent of the minor-road design speed; and then (2) cross and clear the intersection at that same speed. The value of t_c should equal or exceed the appropriate travel time for crossing the major road from a Stop-controlled approach, as

shown in Table IX-10. If the major road is a divided highway with a median wide enough to store the design vehicle for the crossing maneuver, then only crossing of the near lanes need be considered and a departure sight triangle for accelerating from a stopped position in the median should be provided based on Case B1.

Left- and Right-Turn Maneuvers (Case C2)

The length of the leg of the approach sight triangle along the minor road to accommodate left and right turns without stopping (distance a in Figure IX-32A) should be 25 m. This distance is based on the assumption that drivers making left and right turns without stopping will slow to a turning speed of 16 km/h. The leg of the approach sight triangle along the major road (distance b in Figure IX-32B) is similar to the major-road leg of the departure sight triangle for a Stop-controlled intersections in Cases B1 and B2, except that for a Yield-controlled intersection, the travel times in Table IX-9 should be increased by 0.5 sec. The minor-road vehicle requires 3.5 sec to travel from the decision point to the intersection. This 3.5 sec represents additional travel time that is needed at a Yield-controlled intersection (Case C), but is not needed at a Stop-controlled intersection (Case B). However, the acceleration time after entering the major road is 3.0 sec less for a Yield sign than for a Stop sign because the turning vehicle accelerates from 16 km/h rather than from a stop. The net 0.5-sec increase in travel time for a vehicle turning from a Yield-controlled approach is the difference between the 3.5 sec increase in travel time and the 3.0 sec reduction in travel time explained above.

Departure sight triangles like those provided for Stop-controlled approaches (see Cases B1, B2, and B3) should also be provided for Yield-controlled approaches to accommodate minor-road vehicles that stop at the Yield sign to avoid conflicts with major road vehicles. However, since approach sight triangles for turning maneuvers at Yield-controlled are larger than the departure sight triangles used at Stop-controlled intersections, no specific check of departure sight triangles at Yield-controlled intersection should be necessary.

Yield-controlled approaches generally require greater sight distance than Stop-controlled approaches, especially at four-leg Yield-controlled intersections where the sight distance needs of the crossing maneuver must be considered. If sight distance sufficient for Yield control is not available, use of a Stop sign instead of a Yield sign should be considered. In addition, at locations where the recommended sight distance cannot be provided, consideration should be given to installing advisory speed signing on the major road to reduce the speeds of approaching vehicles.

Intersections with Traffic Signal Control (Case D)

At signalized intersections, the first vehicle stopped on each approach should be visible to the driver of the first vehicle stopped on each of the other approaches. Apart from this criterion, there are generally no other approach or departure sight triangles needed for signalized intersections. Indeed, signalization may be an appropriate accident countermeasure for higher volume intersections with restricted sight distance that have experienced a pattern of sight-distance related accidents. However, if the traffic signal is to be placed on two-way flashing operation (i.e., flashing amber on the major-road approaches and flashing red on the minor-road approaches) under off-peak or nighttime conditions, then the appropriate departure sight triangles for Case B, both to the left and to the right, should be provided for the minor-road approaches. In addition, if right turns on a red signal are to be permitted from any approach, then the appropriate departure sight triangle to the left for Case B2 should be provided to accommodate right turns from that approach.

Intersections with All-Way Stop Control (Case E)

At intersections with all-way Stop control, the first stopped vehicle on each approach should be visible to the drivers of the first stopped vehicles on each of the other approaches. There are no other sight distance criteria applicable to intersections with all-way Stop control and, indeed, all-way Stop control may be the best option at a limited number of intersections where the provision of sight distance for other control types cannot be attained.

Left Turns from a Major Road (Case F)

All locations along a major highway from which vehicles are permitted to turn left across opposing traffic, including at-grade intersections and driveways, should have sufficient sight distance to accommodate left-turn maneuvers. Left-turning drivers need sufficient sight distance to decide when it is safe to turn left across the lane(s) used by opposing traffic. Sight distance design should be based on a left turn by a stopped vehicle, since a vehicle that turns left without stopping would need less sight distance. The sight distance along the major road to accommodate left turns is the distance that would be traversed at the design speed of the major-road in the travel time for the appropriate design vehicle given in Table IX-12. The table also contains appropriate adjustment factors for the number of major-road lanes to be crossed by the turning vehicle.

If stopping sight distance has been provided continuously along the major road and if sight distance for Case B (Stop control) or Case C (Yield control) has been provided for each minor-road approach, sight distance will generally be adequate for left turns from the major road. Thus, no separate check of sight distance for Case F is generally required.

However, at three-leg intersections or driveways located on or near a horizontal curve on the major road, the availability of adequate sight distance for left turns from the major road should be checked. In addition, the availability of sight distance for left turns from divided highways should be checked because of the possibility of sight obstructions in the median.

At four-leg intersections on divided highways, opposing vehicles turning left can block a driver's view of oncoming traffic. Figure IX-74, presented later in this chapter, illustrates intersection designs that can be used to offset the opposing left-turn lanes and provide left-turning drivers with a better view of oncoming traffic [the figure referred to was recommended for inclusion in the Green Book in NCHRP Report 375, entitled "Median Intersection Design" (*34*) and is presented as Figure 42 in the main text of this report].

Effect of Skew

When two highways intersect at an angle less than 60 degrees, and when realignment to increase the angle of intersection is not justified, some of the factors for determination of intersection sight distance may need adjustment.

Each of the clear sight triangles described above are applicable to oblique-angle intersections. As shown in Figure IX-33, the legs of the sight triangle will lie along the inter-

Table IX-12. Travel Times Used to Determine the Sight Distance along the Major Road to Accommodate Left Turns from the Major Road (Case F).

Design vehicle	Travel time (sec) at design speed of major road
Passenger car	5.5
Single-unit truck	6.5
Combination truck	7.5

Adjustment for multilane highways:

For left turns that must cross more than one opposing lane, add 0.5 sec for passenger cars and 0.7 sec for trucks for each additional lane to be crossed.

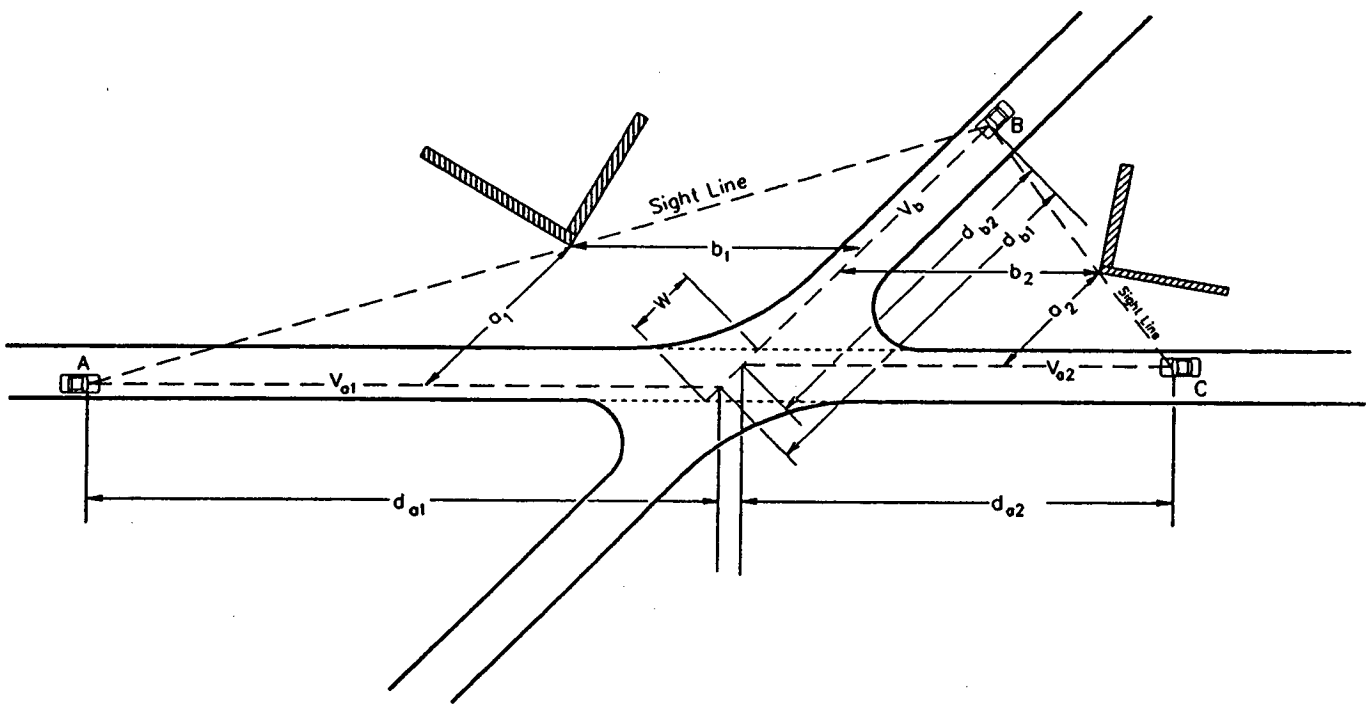


Figure IX-33. Effect of skew on sight distance at intersections.

section approaches and each sight triangle will be larger or smaller than the corresponding sight triangle would be at a right-angle intersection. The area within each sight triangle should be clear of potential sight obstructions as described above.

At an oblique-angle intersection, the length of the travel paths for some turning and crossing maneuvers will be increased. The actual path length for a turning or crossing maneuver can be computed by dividing the total widths of the lanes (plus the median width, where appropriate) to be crossed by the sine of the intersection angle. If the actual path length exceeds the total widths of the lanes to be crossed by 3.6 m or more, then an appropriate number of additional lanes should be considered in applying the adjustment for the number of lanes to be crossed shown in Table IX-9 for Cases B1 and B2 and in Table IX-10 for Case B3. For Case C, the w term in the equation for the minor-road leg of the sight triangle to accommodate the crossing maneuver at a Yield-controlled intersection should also be divided by the sine of the intersection angle to obtain the actual path length.

In the obtuse-angle quadrant of an oblique-angle intersection, the angle between the approach leg and the sight line is often so small that drivers can look across the full sight triangle with only a small head movement. However, in the acute-angle quadrant, drivers are often required to turn their heads considerably to see across the entire clear sight triangle. For this reason, it is recommended that the sight distance criteria for Case A not be applied to oblique-angle intersections and that sight distances at least equal to those for Case B should be provided, whenever possible.

POTENTIAL CHANGES TO RECOMMENDED GREEN BOOK REVISIONS IF THE AASHTO SSD MODEL IS REVISED

Several of the preceding recommendations are based on the current AASHTO SSD model. If AASHTO adopts the revised SSD model and parameter values recommended by Fambro et al. ("Determination of Stopping Sight Distances," Final Report of a forthcoming NCHRP project), the following changes in the revised Green Book text are recommended:

- The driver eye height and object height used in the identification of sight obstructions within clear sight triangles and in the design of vertical curves to provide ISD should both be changed from 1,070 to 1,080 mm above the roadway surface.
- The sight distance values for Case A presented in Table IX-7 should be replaced with the corresponding values from Table 11 in Chapter 2 in the main text of this report.
- The distance along the minor road for Case C1 in Table IX-11 should be replaced with the corresponding values from Table 33 in Chapter 4 in the main text of this report. Similarly, the travel times in Table IX-11 should be replaced with the corresponding values from Table 34.

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Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S.DOT	United States Department of Transportation

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