NCHRP
SYNTHESIS 299

NATIONAL
COOPERATIVE
HIGHWAY
RESEARCH
PROGRAM

Recent Geometric Design Research for Improved Safety and Operations

TRANSPORTATION RESEARCH BOARD

NATIONAL RESEARCH COUNCIL

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NCHRP SYNTHESIS 299

Recent Geometric Design Research for Improved Safety and Operations

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Highway and Facility Design and Highway Operations, Capacity, and Traffic Control

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Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

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The members of the technical committee selected to monitor this project and to review this report were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. The opinions and conclusions expressed or implied are those of the research agency that performed the research, and, while they have been accepted as appropriate by the technical committee, they are not necessarily those of the Transportation Research Board, the National Research Council, the American Association of State Highway and Transportation Officials, or the Federal Highway Administration of the U.S. Department of Transportation.

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.

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PREFACE

A vast storehouse of information exists on nearly every subject of concern to highway administrators and engineers. Much of this information has resulted from both research and the successful application of solutions to the problems faced by practitioners in their daily work. Because previously there has been no systematic means for compiling such useful information and making it available to the entire community, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize useful knowledge from all available sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series reports on various practices, making specific recommendations where appropriate but without the detailed directions usually found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems. The extent to which these reports are useful will be tempered by the user's knowledge and experience in the particular problem area.

FOREWORD

By Staff
Transportation Research Board

This synthesis report will be of interest to roadway geometric design, safety, and operations engineers, researchers, and managers. It reviews and summarizes selected geometric design research published during the 1990s, particularly research with improved safety and operations implications. Information for the synthesis study was collected using an extensive literature review and analysis. A short survey of U.S. transportation agencies was also used to gather additional published information and to identify projects that may not have been included in national databases.

Administrators, engineers, and researchers are continually faced with highway problems on which much information exists, either in the form of reports or in terms of undocumented experience and practice. Unfortunately, this information often is scattered and unevaluated and, as a consequence, in seeking solutions, full information on what has been learned about a problem frequently is not assembled. Costly research findings may go unused, valuable experience may be overlooked, and full consideration may not be given to available practices for solving or alleviating the problem. In an effort to correct this situation, a continuing NCHRP project has the objective of reporting on common highway problems and synthesizing available information. The synthesis reports from this endeavor constitute an NCHRP publication series in which various forms of relevant information are assembled into single, concise documents pertaining to specific highway problems or sets of closely related problems.

This report of the Transportation Research Board provides information on selected research published during the 1990s regarding geometric design and roadways, with emphasis on research with safety and operations implications. In part, the incentive for this study was for an updating of AASHTO's *A Policy on Geometric Design of Highways and Streets*, more commonly referred to as the *Green Book*, the last version of which was published in 1990 for English units and in 1994 for metric units. The results of this review are presented in agreement with the primary sections of the *Green Book*; that is, design controls and criteria, elements of design, cross sections, intersections, and interchanges. Because this is such a broad topic, and correspondingly a longer than usual synthesis, the results will be posted in a searchable format on the Transportation Research Board's website at http://www4.trb.org/trb/onlinepubs.nsf/web/nchrp_synthesis.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the available information was assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the author's research in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that now at hand.

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Information on current practice was provided by many highway and transportation agencies. Their cooperation and assistance are appreciated.

RECENT GEOMETRIC DESIGN RESEARCH FOR IMPROVED SAFETY AND OPERATIONS

SUMMARY

During the last decade, there has been considerable research on all aspects of geometric design affecting how roadways are designed, how they operate, and, ultimately, the safety of these facilities. A limitation to the potential application of this research is the sheer volume of information that was published during this period. This synthesis of recent research was developed to provide national, state, and local geometric design policymakers with a summary of geometric design research published in the 1990s, particularly research with safety and operational implications. The review of the literature identified a number of key findings that can have an impact on current practice and methodology, thus leading to recommended changes in design or practice modifications of current guidelines. The findings within the synthesis are presented in groups similar to key chapters within AASHTO's *A Policy on Geometric Design of Highways and Streets*.

- Design Controls and Criteria—Revisions to a design manual should reference or incorporate information, as appropriate, from several new publications: *Older Driver Highway Design Handbook, Highway Capacity Manual* (published in 2000), the *Guide for the Development of Bicycle Facilities*, *Traffic Safety Toolbox: A Primer on Traffic Safety* (and other documents on safety), *Access Management Guidelines for Activity Centers*, *Impacts of Access Management Techniques*, *Driveway and Street Intersection Spacing, HOV Systems Manual*, the *Design and Safety of Pedestrian Facilities*, and the Public Rights-of-Way Access Advisory Committee's final report to the U.S. Access Board (*Building a True Community*). Several publications have suggested that when revising a design manual the revisions should include design consistency concepts and designers should be encouraged to "think about the pedestrian."

- Elements of Design—Stopping sight distance and superelevation have been topics of recent NCHRP studies. These studies have developed extensive recommendations for changes. Other topics examined in the literature include truck and passenger car speeds on horizontal curves, the use of zero-length and minimum-length vertical curves, passing lanes, climbing lanes, and the concept of rating designs as good, fair, and poor based upon the speed differential between consecutive sections or between the design speed and the operating speed.

- Cross Sections—Recent publications have discussed the tradeoffs of using shoulders and/or narrowing travel lanes on freeways, methods for selecting alternative median treatments, tradeoffs for alternative uses of a cross section, the use of curbs and gutters on suburban highways, and the relationship between safety and cross-sectional elements.

- Intersections—Topics discussed within the literature on intersections include alternative intersection designs (e.g., roundabouts); appropriate median widths at intersections; intersection sight distance; corner clearances; driveway vertical curves; determination of whether a left-turn lane should be used and its appropriate length; use of offsetting opposing left-turn lanes, triple left-turn lanes, and right-turn lanes; and available crash models.

- Interchanges—The literature provides characteristics of single-point urban interchanges along with comparisons of different interchange types. Recommendations relating ramp design policy to the anticipated operating speed of the ramp have also been presented. Other areas discussed included two-lane loop ramps; pedestrian needs at expressway ramps; and design of urban interchanges, entrance ramp meters, high-occupancy vehicle bypass lanes, and the at-grade intersection portion of an interchange ramp terminal.

CHAPTER ONE

INTRODUCTION

BACKGROUND

The American Association of State Highway and Transportation Official's (AASHTO) *A Policy on Geometric Design of Highways and Streets* (commonly referred to as the *Green Book*) presents the national policy for geometric design. This document has been updated on several occasions with new research findings. At the time of this study, the most recent versions had been published in 1990 (*1*) in English units and in 1994 (*2*) for metric units. During the 1990s, there were a considerable number of research projects on all aspects of geometric design including how roadways are designed, how they operate, and, ultimately, the safety of the facilities. A limitation to the potential application of the research was the sheer volume of information that had been published. A synthesis of recent research was needed. The NCHRP Synthesis project (20-05) funded a study to develop such a synthesis, which resulted in this document, *Recent Geometric Design Research for Improved Safety and Operations.*

This synthesis study reviewed and summarized the geometric design research published during the 1990s, particularly research with safety and operational implications, and addressed the following areas:

- Design speed,
- Controls and criteria (e.g., definitions, vehicles, users),
- Horizontal alignment,
- Vertical alignment,
- Cross sections (including roadside elements),
- Intersections,
- Interchanges,
- Access management, and
- Design consistency.

STUDY OBJECTIVE

The objective of the study was to review and selectively summarize geometric design research published during the 1990s, particularly research with safety and operational implications. The study used two approaches: (1) a review of the literature contained in national databases, and (2) a questionnaire to states to assist in identifying projects that may not have been included in national databases.

The literature review represented the majority of the effort for this synthesis study. The Transportation Research Information Services (TRIS) was used to identify potential papers and reports published within the previous 10 years. Other sources for information on research activities included the Synthesis Study Topic Panel, the TRB Committees on Geometric Design (A2A02) and Operational Effects of Geometrics (A3A08), and findings from the survey. The relevant literature was reviewed and key findings were summarized.

ORGANIZATION OF REPORT

This synthesis consists of the introduction, five chapters that focus on the findings from the literature, a summary chapter, and one appendix. The introduction provides an overview of the project, starting with a brief discussion on the project background and project objective. Chapters 2–6 provide summaries and pertinent tables and figures of the findings from the literature. These chapters were arranged to match the presentation of material in the *Green Book* (*1,2*). They include the following:

- Chapter 2—Design Controls and Criteria
- Chapter 3—Elements of Design
- Chapter 4—Cross Sections
- Chapter 5—Intersections
- Chapter 6—Interchanges

The final chapter of the report provides a summary of the key findings from the literature along with issues to be considered in future editions of design manuals. The single appendix (Appendix A) presents the findings from the survey distributed to the states.

CHAPTER TWO

DESIGN CONTROLS AND CRITERIA

DESIGN VEHICLES

Gattis and Howard (*3*) generated design vehicle characteristics for large or full-size school buses. The objectives of the study included identifying which full-size school buses are being used by larger school districts and measuring the bus path and turning radii for 90- and 180-degree turns at crawl speeds. From the bus manufacturers' specifications and test results, the worst cases (e.g., longest and widest) of each dimension and turning radii were combined into a hybrid design vehicle. Table 1 compares the bus dimensions developed with those of the AASHTO SU and BUS design vehicles. The comparison showed that the school bus turn radius was less than that of the AASHTO vehicles. Therefore, using AASHTO SU or BUS vehicles as surrogate design vehicles for school buses (as several state transportation agencies do) is conservative and, for the most part, leads to designing a larger turning area than is actually needed for school bus use. However, the SU or BUS templates do not depict the "kick out" that occurs at the beginning of a turn and, in this respect, they produce an inadequate turning area. A kick out is when the rear corner of the vehicle would swing out past the path or trace made by the front of the bus. This action could cause the rear of the bus to move into the adjoining lane during a left or right turn. The authors recommended that when dealing with vehicles with large rear overhangs, the designer should be cognizant of the problem.

Harkey et al. (*4*) identified research studies needed to understand how longer-combination vehicles (LCVs) operate in order to better accommodate them through geometric designs or regulate them through more stringent laws and better enforcement. LCVs include Rocky Mountain doubles, turnpike doubles, and triples. LCVs handle and perform differently from tractor semitrailers or twin trailers because of their increased lengths and weights. These differences in handling and performance may jeopardize the safety of the LCV as well as other vehicles on the roadway. Several of the LCV's operational characteristics are believed to have an impact on transportation safety and the relationship of these characteristics to geometric design. The authors stated that there is a clear need to conduct additional research to further evaluate LCV operations. Suggested research included:

- Operations on rural roads with severe horizontal and vertical curvature,
- Operations on rural roads with passing zones and moderate horizontal curvature,
- Operations on congested freeways, and
- Operations at rural and urban intersections.

Miaou and Lum (*5*) used a Poisson regression model to evaluate the effects of highway geometric design on truck accident involvement rates and to estimate and quantify the uncertainties of the expected reductions in truck accident involvements from various improvements in highway geometric design. Five years of highway geometric, traffic, and truck accident data for rural Interstate highways in Utah (1985–1989) were used. The analyses developed predictions for the number of truck accident reductions caused by improvements in horizontal curvature, vertical grade, and paved inside shoulder width of a road section. Tables 2 and 3 are the expected reductions caused by an improvement in one geometric design element and two geometric design elements, respectively.

TABLE 1
RECOMMENDED DESIGN VEHICLES COMPARED WITH SU AND BUS (*3*)

Design Vehicle Type	Symbol	Overall Width (ft)/m	Overall Length (ft)/m	Overhang Front (ft)/m	Overhang Rear (ft)/m	Wheel-Base (ft)/m	Turning Radius of Wheel Outer Front (ft)/m	Turning Radius of Wheel Inner Rear (ft)/m
Single-Unit Truck	SU	(8.5) 2.6	(30.0) 9.1	(4.0) 1.2	(6.0) 1.8	(20.0) 6.1	(42.0) 12.8	(27.8) 8.5
Single-Unit Bus	BUS	(8.5) 2.6	(40.0) 12.2	(7.0) 2.1	(8.0) 2.4	(25.0) 7.6	(42.0) 12.8	(24.4) 7.4
School Bus Type C	SB-C	(8.0) 2.44	(36.4) 11.1	(2.8) 0.8	(12.1) 3.7	(21.5) 6.6	(38.9) 11.8	(25.2) 7.7
School Bus Type D	SB-D	(8.0) 2.44	(40.0) 12.2	(7.0) 2.1	(10.0) 3.2	(23.0) 7.0	(40.0) 12.2	(25.2) 7.7
School Bus Type D alt. max.	SB-D			(8.0) 2.4	(13.8) 4.2			

TABLE 2
EXPECTED REDUCTIONS IN TRUCK ACCIDENT INVOLVEMENTS ON A RURAL INTERSTATE ROAD SECTION AFTER AN IMPROVEMENT IN ONE GEOMETRIC DESIGN ELEMENT (*5*)

Length of Original Curve, mi (km)	Horizontal Curvature (HC) in degrees/100 ft (30.5 m) arc: for $2^o \le HC \le 12^o$				
	Reduce 1^o	Reduce 2^o	Reduce 3^o	Reduce 4^o	Reduce 5^o
0.10 (0.2)	10.6% ± 2.5%	20.1% ± 4.5%	28.6% ± 6.0%	36.2% ± 7.2%	43.0% ± 8.1%
0.25 (0.4)	13.7% ± 1.9%	25.5% ± 3.3%	35.7% ± 4.2%	44.5% ± 4.9%	52.1% ± 5.3%
0.50 (0.8)	18.6% ± 2.7%	33.8% ± 4.4%	46.1% ± 5.4%	56.1% ± 5.8%	64.3% ± 6.0%
0.75 (1.2)	23.2% ± 4.3%	41.1% ± 6.6%	54.8% ± 7.7%	65.3% ± 8.0%	73.4% ± 7.8%
≤1.00 (1.6)	27.6% ± 5.8%	47.6% ± 8.6%	62.1% ± 9.6%	72.5% ± 9.5%	80.1% ± 9.0%
Length of Original Grade, mi (km)	**Vertical Grade (VG): for 2% < VG < 9%**				
	Reduce 1%	Reduce 2%	Reduce 3%	Reduce 4%	Reduce 5%
0.10 (0.2)	7.8% ± 3.1%	15.0% ± 5.7%	21.6% ± 7.9%	27.7% ± 9.7%	33.4% ± 11.3%
0.50 (0.8)	9.0% ± 2.5%	17.3% ± 4.6%	24.7% ± 6.3%	31.5% ± 7.7%	37.7% ± 8.8%
1.00 (1.6)	10.6% ± 2.1%	20.0% ± 3.7%	28.5% ± 5.0%	36.0% ± 5.9%	42.8% ± 6.7%
≤2.00 (3.2)	13.5% ± 2.1%	25.3% ± 3.6%	35.4% ± 4.6%	44.2% ± 5.4%	51.7% ± 5.8%
	Paved Inside Shoulder Width (ISH) per Direction: for ISH ≤ 12 ft				
	Increase 1 ft (0.3 m)	Increase 2 ft (0.6 m)	Increase 3 ft (0.9 m)	Increase 4 ft (1.2 m)	Increase 5 ft (1.5 m)
	8.2% ± 4.2%	15.7% ± 7.7%	22.7% ± 10.7%	29.0% ± 13.2%	34.9% ± 15.4%

TABLE 3
EXPECTED REDUCTIONS IN TRUCK ACCIDENT INVOLVEMENTS ON A RURAL INTERSTATE ROAD SECTION AFTER AN IMPROVEMENT IN TWO GEOMETRIC DESIGN ELEMENTS (*5*)

Length of Original Curve (LHC) = 0.10 mi (0.2 km) and Length of Original Grade (LVH) = 0.50 mi (0.8 km)					
Vertical Grade (VG): for 2% < VG < 9%	Horizontal Curvature (HC) in degrees/100 ft (30.5 m) arc: $2^o \le HC \le 12^o$				
	Reduce 1^o	Reduce 2^o	Reduce 3^o	Reduce 4^o	Reduce 5^o
Reduce 1%	18.7% ± 3.1%	27.3% ± 4.4%	35.0% ± 5.6%	42.0% ± 6.6%	48.1% ± 7.4%
Reduce 2%	26.0% ± 4.5%	33.9% ± 5.0%	40.9% ± 5.8%	47.2% ± 6.4%	52.8% ± 6.9%
Reduce 3%	32.7% ± 5.8%	39.9% ± 5.9%	46.3% ± 6.2%	52.0% ± 6.5%	57.1% ± 6.9%
Reduce 4%	38.8% ± 7.0%	45.3% ± 6.7%	51.1% ± 6.7%	56.3% ± 6.7%	61.0% ± 6.9%
Reduce 5%	44.3% ± 7.9%	50.3% ± 7.4%	55.5% ± 7.1%	60.3% ± 7.0%	64.5% ± 6.9%
Length of Original Curve (LHC) = 0.10 mi (0.2 km)					
Paved Inside Shoulder Width per Direction (ISH): for ISH ≤ 12 ft (3.7 m)	Horizontal Curvature (HC) in degrees/100 ft (30.5 m) arc: for $2^o \le HC \le 12^o$				
	Reduce 1^o	Reduce 2^o	Reduce 3^o	Reduce 4^o	Reduce 5^o
Increase 1 ft (0.3 m)	18.0% ± 4.4%	26.7% ± 5.3%	34.5% ± 6.3%	41.4% ± 7.1%	47.6% ± 7.8%
Increase 2 ft (0.6 m)	24.7% ± 7.2%	32.7% ± 7.3%	39.9% ± 7.5%	46.2% ± 7.9%	52.0% ± 8.2%
Increase 3 ft (0.9 m)	30.9% ± 9.8%	38.2% ± 9.3%	44.8% ± 9.0%	50.7% ± 8.9%	55.9% ± 8.9%
Increase 4 ft (1.2 m)	36.6% ± 12.0%	43.3% ± 11.1%	49.3% ± 10.5%	54.7% ± 10.0%	59.5% ± 9.7%
Increase 5 ft (1.5 m)	41.8% ± 13.8%	48.0% ± 12.7%	53.5% ± 11.8%	58.4% ± 11.1%	62.8% ± 10.5%

The ability of the roadway system to accommodate large trucks is constrained by the geometric design of key features, including horizontal curves, interchange ramps and interchange ramp terminals, at-grade intersections, and steep grades. Part of an FHWA study was used to determine the ability of the current roadway system to accommodate trucks of the future. Data on the distribution of radii for horizontal curves and vertical grades on mainline roadways were available from the FHWA Highway Performance Monitoring System. Data on actual geometrics of interchange ramps and at-grade intersections were obtained with the assistance of nine state highway agencies (split between five regions). For all states combined, data were obtained for 436 interchanges and 379 at-grade intersections. All of the intersections considered were locations on known truck routes where trucks were considered likely to turn. Table 4 summarizes some of the findings from the study (*6*).

DRIVER PERFORMANCE

The FHWA sponsored a research project that developed specific recommendations for accommodating older drivers. In 1995, the 65 and older age group in the United States numbered 33.5 million and is predicted to grow to more than 36 million by 2005 and exceed 50 million by 2020, accounting for roughly one-fifth of the driving-age population in the country. The research produced an *Older*

TABLE 4
GEOMETRIC DESIGN DISTRIBUTIONS (*6*)

Radii of Horizontal Curves on Mainline Roadways	• Most of the mainline curves have a degree of curvature between 0.0 and 2.4 for all highway types and regions, with the only exception being rural two-lane roadways. • A large percentage of the curves on rural two-lane roadways have a degree of curvature between 0.0 and 2.4, but the total mileage of curves with degree of curvature between 2.5 and 6.9 is substantially higher compared with the other highway types.
Grades on Mainline Roadways	• Grades steeper than 6.4 percent, which slow trucks the most, make up only a very small portion of the highway network. • Grades steeper than 6.4 percent constitute only 1.4 percent of the rural two-lane mileage, only 1.4 percent of rural multilane highway mileage, only 0.8 percent of urban arterial mileage, 0.2 percent of rural freeway mileage, and 0.2 percent of urban freeway mileage.
Radii of Horizontal Curves on Interchange Ramps	• Only 5 percent of the surveyed ramps have radii of 100 ft (30 m) or less, and approximately 20 percent of rural ramps and 30 percent of urban ramps have radii of 250 ft (75 m) or less. • For rural ramps on which the sharpest horizontal curve had a radius of 250 ft (75 m) or less, widening for 35 percent of the ramps was classified as very difficult, 18 percent as moderately difficult, and 47 percent as feasible. • For urban ramps, the comparable percentages were 42 percent very difficult, 19 percent moderately difficult, and 39 percent feasible.
Curb Return Radii at Ramp Terminals	• In general, approximately 10 percent of rural ramp terminals and 20 percent of urban ramp terminals have curb return radii of 40 ft (12 m) or less. • For curb return radii at rural ramp terminals with radii of 40 ft (12 m) or less, widening for 8 percent of the ramps was classified as very difficult, 11 percent were classified as moderately difficult, and 81 percent were classified as feasible. • For urban ramps, the comparable percentages were 18 percent very difficult, 29 percent moderately difficult, and 53 percent feasible.
Curb Return Radii for At-Grade Intersections	• Approximately 35 percent of rural at-grade intersections and 45 percent of urban at-grade intersections have radii of 40 ft (12 m) or less. • For rural intersections, widening was classified as very difficult for 18 percent, moderately difficult for 39 percent, and feasible for 43 percent. • For urban intersections, the comparable percentages were 44 percent very difficult, 30 percent moderately difficult, and 27 percent feasible.

Driver Highway Design Handbook (*7*), which is intended to supplement standard design manuals for practitioners. Another report (*8*) contains just the recommendations from the larger *Handbook*. The authors note that the recommendations do not constitute a new standard of required practice. The recommendations provide guidance that is grounded in an understanding of older drivers' needs and capabilities. An example of a recommendation contained in the *Handbook* is shown here.

Street Name Signage

- To accommodate the reduction in visual acuity associated with increasing age, a minimum letter height of 6 in. (150 mm) is recommended for use on post-mounted street name signs.
- The use of overhead-mounted street name signs with minimum letter heights of 8 in. (200 mm) is recommended at major intersections.
- Wherever an advance intersection warning sign is erected (e.g., W2-1, W2-2, W2-3, W2-4), it is recommended that it be accompanied by a supplemental street name sign.
- The use of redundant street name signing for major intersections is recommended, with an advance street name sign placed upstream of the intersection at a midblock location, and an overhead-mounted street name sign posted at the intersection. Wherever practical, the midblock sign should be mounted overhead.
- When different street names are used for different directions of travel on a crossroad, the names should be separated and accompanied by directional arrows on both midblock and intersection street name signs.

Lerner (*9*) evaluated the adequacy of the perception–reaction time (PRT) for intersection sight distance, stopping sight distance, and decision sight distance, especially for older drivers. He also looked at gap acceptance. The study found that differences in PRT between age groups were trivial and the current AASHTO PRT design assumptions for intersection and stopping sight distances appear adequate for the full range of drivers. For decision sight distance, older drivers did show longer PRTs than younger drivers. The experiment for gap/lag acceptance also observed age differences—older drivers generally required somewhat longer gaps or lags in traffic before they would be willing to make a turning or crossing maneuver.

An analysis of freeway accidents was conducted using a total of 36,142 crashes for drivers aged 31 to 45 and 4,155 crashes for drivers aged 66 and older from 5 states over a 5-year period (*10*). The results of the analysis concluded that

- Older drivers were over involved in accidents in which they were merging or changing lanes.

- Older drivers were cited twice as often as younger drivers for all accidents and five times as often for those accidents involving a lane-change maneuver.
- Older drivers also appear to be over involved in run-off-road and single-vehicle accidents both to the left and to the right.
- Older drivers appear to be over involved in both single-vehicle and multiple-vehicle accidents during daylight hours, clear/cloudy weather conditions, and on dry road surfaces when compared with the younger age group. These results are most likely due to exposure differences, reflecting the fact that older drivers conduct a larger percentage of their driving under these "good" conditions as compared with younger drivers.

TRAFFIC CHARACTERISTICS

A Texas study reviewed the relationship between design speed, operating speed, and posted speed (*11*). Design speed is used in selecting the vertical and horizontal elements for new roadways, whereas speed limits are based on a statistical analysis of individual vehicular speeds. At some locations, the posted speed limit, based on an 85th percentile speed, exceeds the roadway's design speed. When posted speed exceeds design speed, liability concerns arise even though the drivers can safely exceed the design speed. Research conducted in the Texas project clearly indicated that department of transportation (DOT) officials are concerned with the potential liability; however, only a few of the respondents to surveys and interviews had actually experienced a lawsuit relevant to the design speed–posted speed issue. The respondents indicated that the primary liability concern rests with the current AASHTO definition of design speed. If the definition were changed to reflect its actual meaning, then liability concerns would be reduced substantially.

An ongoing NCHRP study on design speed and operating speed (*12*) is reevaluating current procedures, including how speed is used as a control in existing policy and guidelines, and is evaluating alternative methods for design. The project team also participated in discussions regarding changing definitions for speed terms, including design speed.

HIGHWAY CAPACITY

A new edition of the *Highway Capacity Manual* (*HCM*) was published in 2000 (*13*). It provides transportation practitioners and researchers with a consistent system of techniques for the evaluation of the quality of service on highway and street facilities. The *HCM* does *not* set policies regarding a desirable or appropriate quality of service for various facilities, systems, regions, or circumstances. Its objectives include providing a logical set of methods for assessing transportation facilities, assuring that practitioners have access to the latest research results, and presenting sample problems. The fourth edition of the *HCM* is intended to provide a systematic and consistent basis for assessing the capacity and level of service for elements of the surface transportation system and also for systems that involve a series or a combination of individual facilities. The manual is the primary source document embodying research findings on capacity and quality of service and presenting methods for analyzing the operations of streets and highways and pedestrian and bicycle facilities.

ACCESS CONTROL

NCHRP Report 348 (*14*) reviewed the overall concept of access management and current practices and set forth basic policy, planning, and design guidelines. The report covered (1) legal and institutional bases for controlling access, (2) access permit procedures and traffic impact studies, (3) access categories (level) and spacing standards, and (4) design concepts and criteria. Access management was defined as the "process that provides or manages access to land development while simultaneously preserving the flow of traffic on the surrounding road system in terms of safety, capacity needs, and speed." The design concepts and criteria involve (1) limiting the number of conflict points, (2) separating conflict areas, (3) reducing acceleration and deceleration impacts at access points, (4) removing turning vehicles from through travel lanes, (5) spacing major intersections to facilitate progressive travel speeds along arteries, and (6) providing adequate on-site storage.

NCHRP Report 420 (*15*) reviewed the impacts of access management techniques. The following is a summary of key findings:

- The spacing of traffic signals, in terms of their frequency and uniformity, governs the performance of urban and suburban highways. It is one of the most important access management techniques.
- Accident rates (per million vehicle-miles of travel or per million vehicle-kilometers of travel) rise as traffic signal density increases. An increase from two to four traffic signals per mile (1.2 to 2.5 traffic signals per kilometer) resulted in an approximately 40 percent increase in accidents along highways in Georgia and an approximately 150 percent increase along US 41 in Lee County, Florida.
- Each traffic signal per mile added to a roadway reduces speed approximately 2 to 3 mph (3.2 to 4.8 km/h). Table 5 lists the percent increase in travel time for increasing numbers of signals.
- Spacing of unsignalized access points is also significant in determining the safety and performance of

TABLE 5
PERCENT INCREASES IN TRAVEL TIMES AS SIGNAL DENSITY INCREASES (*15*)

Signals per Mile (km)	Increase in Travel Time (%)*
2 (1.2)	0
3 (1.9)	9
4 (2.5)	16
5 (3.1)	23
6 (3.7)	29
7 (4.3)	34
8 (5.0)	39

*Compared with 2 signals per mile (1.2 signals per kilometer).

TABLE 6
REPRESENTATIVE ACCIDENT RATES BY ACCESS DENSITY—URBAN AND SUBURBAN AREAS (*15*)

Unsignalized Access Points per Mile	Accident Rates (accidents per million VMT)			
	Signalized Access Points per Mile			
	≤ 2	2.01–4.00	4.01–6.00	>6
≤ 20	2.6	3.9	4.8	6.0
20.01–40	3.0	5.6	6.9	8.1
40.01–60	3.4	6.9	8.2	9.1
>60	3.8	8.2	8.7	9.5
All	3.1	6.5	7.5	8.9

Note: 1 mi = 1.61 km; VMT = vehicle-miles traveled.

TABLE 7
REPRESENTATIVE ACCIDENT RATES BY TYPE OF MEDIAN (*15*)

Total Access Points per Mile*	Accident Rates (accidents per million VMT)		
	Median Type		
	Undivided	Two-Way Left-Turn Lane	Nontraversable Median
Urban and Suburban Areas			
≤ 20	3.8	3.4	2.9
20.01–40	7.3	5.9	5.1
40.01–60	9.4	7.9	6.8
>60	10.6	9.2	8.2
All	9.0	6.9	5.6
Rural			
≤ 15	2.5	1.0	0.9
15.01–30	3.6	1.3	1.2
>30	4.6	1.7	1.5
All	3.0	1.4	1.2

Note: 1 mi = 1.61 km; VMT = vehicle-miles traveled.
* Includes both signalized and unsignalized access points.

urban and suburban highways and should be given due consideration. Also, potentially high-volume unsignalized access points should be placed where they conform to traffic signal progression requirements.

- Representative accident rates by access frequency, median type, and traffic signal density are summarized in Table 6 for urban and suburban areas. Table 7 shows how accident rates rise as the total access points per mile increases as a function of the median treatment.
- The three factors that influence the desired access separation distances are safety, operations, and roadway access classification. Direct property access along strategic and principal arterials should be discouraged; however, where access must be provided, adequate spacing should be established to maintain safety and preserve movement. "Spillback" is defined as a right-lane through vehicle being influenced to or beyond a driveway upstream of the analysis driveway. Spillback occurs when the influence length is greater than the driveway spacing minus the driveway width. The spillback rate represents the percentage of right-lane through vehicles that experience this occurrence. Table 8 provides access separation distances for spillback rates of 5 to 20 percent.

TABLE 8
ACCESS SEPARATION DISTANCE ON THE BASIS OF SPILLBACK RATE (*15*)

Posted Speed mph (km/h)	Spillback Rate [ft (m)]*			
	5%	10%	15%	20%
30 (48.3)	335 (102.2)	265 (80.8)[a]	210 (64.1)[b]	175 (53.4)[c]
35 (56.4)	355 (108.3)	265 (80.8)[a]	210 (64.1)[b]	175 (53.4)[c]
40 (64.4)	400 (122.0)	340 (103.7)	305 (93.0)	285 (86.9)
45 (72.5)	450 (137.3)	380 (115.9)	340 (103.7)	315 (96.1)
50 (80.5)	520 (158.6)	425 (129.6)	380 (115.9)	345 (105.2)
55 (88.6)	590 (180.0)	480 (146.4)	420 (128.1)	380 (115.9)

* Spillback rate is based on an average of 30 to 60 right turns per driveway. Spillback occurs when a right-lane through vehicle is influenced to or beyond a driveway upstream of the analysis driveway. The spillback rate represents the percentage of right-lane through vehicles experiencing this occurrence.
[a]Based on 20 driveways per mi (12.4 driveways per km).
[b]Based on 25 driveways per mi (15.5 driveways per km).
[c]Based on 30 driveways per mi (18.6 driveways per km).

- Corner clearances represent the minimum distances that should be required between intersections and driveways along arterial and collector streets. Values assembled from various states, counties, and cities ranged from 16 to 235 ft (4.9 to 71.7 m). Eight case studies indicated that

  – Definition of corner clearance distances varied among locations;
  – Distances ranged from 2 to 250 ft (0.6 to 76.3 m);
  – Queuing or spillback across driveways was perceived as the most pervasive problem, making it difficult to turn left into or out of a driveway;
  – Roadway widening to increase capacity sometimes reduces corner clearances;
  – Placing driveways too close to intersections correlates with higher accident frequencies—sometimes as many as one-half of all accidents involved are driveway-related;
  – Corner clearances are limited by the property frontage available; and
  – Improving or retrofitting minimum corner driveway distances is not always practical, especially in built-up areas.

- Selecting a median alternative depends on factors related to policy, land use, and traffic. These factors include:

  – Access management policy for and access class of the roadway under consideration;
  – Type and intensities of the adjacent land use;
  – Supporting street system and the opportunities for rerouting left turns;
  – Existing driveway spacings;
  – Existing geometric design and traffic control features (e.g., proximity of traffic signals and provisions for left turns);
  – Traffic volumes, speeds, and accidents; and
  – Costs associated with roadway widening and reconstruction.

A procedure for evaluating and selecting median treatments is detailed in *NCHRP Report 395* (*16*) and summarized in *NCHRP Report 420* (*15*).

- The treatment of left turns is a major access management concern. A synthesis of safety experience indicates that the removal of left turns from through traffic lanes reduced accident rates by approximately 50 percent (the range was 18 to 77 percent).
- Providing access to the far side (or opposing direction) of a roadway only through U-turns reduces conflicts and improves safety. U-turns result in a 20 percent accident rate reduction by eliminating direct left turns from driveways and a 35 percent reduction when the U-turns are signalized. Roadways with wide medians and "directional" U-turn crossovers have roughly one-half of the accident rates of roads with two-way left-turn lanes. U-turns, coupled with two-phase traffic signal control, result in an approximately 15 to 20 percent gain in capacity over conventional intersections with dual left-turn lanes and multiphase traffic signal control.
- Access spacing between interchanging arterial roadways ranges from 100 to 700 ft (30.5 to 213.5 m) in urban areas and 300 to 1,000 ft (91.5 to 305.0 m) in rural areas. These distances are usually less than the access spacing needed to ensure good traffic signal progression and to provide adequate weaving and storage for left turns.

TRB Research Circular 456, "Driveway and Street Intersection Spacing" (*17*), notes that access points are the main source of accidents and congestion and that driveway and intersection location and spacing directly affect the safety and functional integrity of streets and highways. The *Circular* is a "compilation of the contemporary practice that illustrates the basic considerations for spacing standards and guidelines and that describes current state, county, and local spacing requirements." Table 9 lists the recommendations from the *Circular* on geometric, traffic control, and spacing requirements by functional classes of roads. Table 10

TABLE 9
FUNCTIONAL ROUTE CLASSIFICATION (*17*)

Classification	Freeway and Expressway	Primary Arterial	Secondary Arterial	Collector	Local
Function	Traffic movement	Intercommunity and intrametro area; Primary—traffic movement and Secondary—land access	Primary—intercommunity, intrametro area, traffic movement Secondary—land access	Collect/distribute traffic between local streets and arterial system; Secondary—lane access, Tertiary—interneighborhood traffic movement	Land access
Typical Percentage of Surface Street System Mileage	NA	5–10	10–20	5–10	60–80
Continuity	Continuous	Continuous	Continuous	Not necessarily continuous; should not extend across arterials	None
Spacing (miles)	4	1 to 2	0.5 to 1	0.5 or less	As needed
Typical Percentage of Surface Street System Vehicle-Miles Carried	NA	40–65	25–40	5–10	10–30
Direct Land Access	None	Limited—major generators only	Restricted—some movements may be prohibited; number and spacing of driveways controlled	Safety controls; limited regulation	Safety controls only
Minimum Roadway Intersection Spacing	1 mile	0.5 mile	0.25 mile	300 ft	300 ft
Speed Limit (mph)	45–55	35–45 in fully developed areas	30–35	25–30	25
Parking	Prohibited	Prohibited	Generally prohibited	Limited	Permitted
Comments	Supplements capacity of arterial street system and provides high-speed mobility		Backbone of street system	Through traffic should be discouraged	Through traffic should be discouraged

Note: 1 mi = 1.61 km; NA = not available.

TABLE 10
OPTIMUM SIGNALIZED INTERSECTION SPACING NEEDED TO ACHIEVE EFFICIENT TRAFFIC PROGRESSION AT VARIOUS SPEEDS AND CYCLE LENGTHS (*17*)

Cycle Length (sec)	Speed mph (km/h)						
	25 (40.3)	30 (48.3)	35 (56.4)	40 (64.4)	45 (72.5)	50 (80.5)	55 (88.6)
	Distance in feet (meters)						
60	1,100 (335.5)	1,320 (402.6)	1,540 (469.7)	1,760 (536.8)	1,980 (603.9)	2,200 (671.0)	2,430 (741.2)
70	1,280 (390.4)	1,540 (469.7)	1,800 (549.0)	2,050 (625.3)	2,310 (704.6)	2,500 (762.5)	2,820 (860.1)
80	1,470 (448.4)	1,760 (536.8)	2,050 (625.3)	2,350 (716.8)	2,640 (805.2)	2,930 (893.7)	3,220 (982.1)
90	1,630 (497.2)	1,980 (603.9)	2,310 (704.6)	2,640 (805.2)	2,970 (905.9)	3,300 (1006.5)	3,630 (1170.2)
120	2,200 (671.0)	2,640 (805.2)	3,080 (939.4)	3,520 (1073.6)	3,960 (1207.8)	4,400 (1342.0)	4,840 (1476.2)
150*	2,750 (838.8)	3,300 (1006.5)	3,850 (1174.3)	4,400 (1342.0)	4,950 (1509.8)	5,500 (1677.5)	6,050 1845.3)

Note: 1 mph = 1.61 km/h; 1 ft = 0.305 m.
*Represents maximum cycle length for actuated signal if all phases are fully used.

TABLE 11
SUMMARY OF MINIMUM UNSIGNALIZED ACCESS SPACING BY SPEED FOR VARIOUS CRITERIA (*17*)

Criteria	Posted Speed (mph)								
	20	25	30	35	40	45	50	55	60
Stopping Sight Distance	120	165	220	275	340	410	485	565	655
Length of Turn Lane: Turning Traffic to Leave Through Lane with a Speed Differential of:									
≤10 mph (16.1 km/h)					490	590	700	820	950
≤15 mph (24.2 km/h)				390	390	490	590	700	820
≤20 mph (32.2 km/h)			320	320	320	390	490	590	700
Minimize Right-Turn Conflict Overlap			100	150	200	300			
Intersection Sight Distance Through Traffic Reduces Speed by 15%	230	300	375	46	575	700	850	1,000	1,150
Maximum Egress Capacity	120	190	320	450	620	860	1,125	1,500	1,875

Note: Values are given in feet (1 ft = 0.305 m).

lists the optimum signalized intersection spacing needed to achieve efficient traffic progression at various speeds and cycle lengths. Table 11 summarizes minimum unsignalized access spacing by speed for various criteria. The *Circular* concludes that spacing criteria should be keyed to the functional classification of the road system, with the more restrictive standards established for a higher type of road. Signalized intersection spacing should maintain maximum bandwidths in each direction of travel at different travel speeds. There is less consensus (and a great need for research) regarding unsignalized intersection spacing and corner clearance.

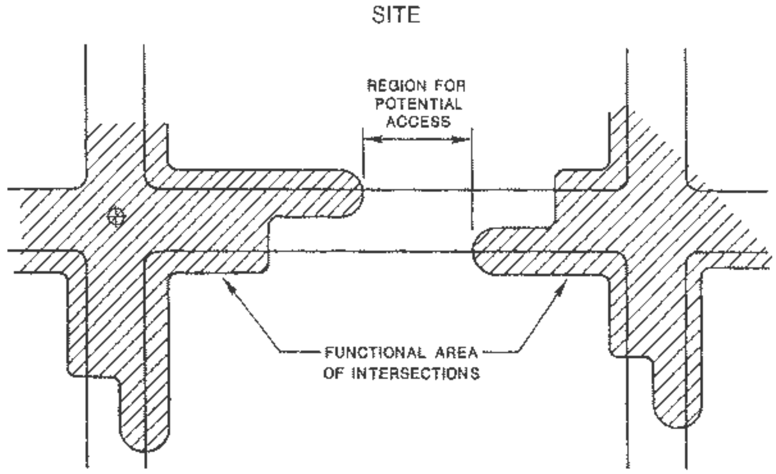
Stover (*18*) argues that the logic of functional design suggests that the urban arterial street design process should begin with the most important intersections and then, in turn, consider intersections that are successively lower in the functional hierarchy. Major issues relative to access control in urban arterial intersection design are listed here.

- Spacing of signalized intersections (private access as well as public streets) so that efficient traffic movement can be achieved on the arterial streets in both peak and off-peak periods.
- Establishment of a functional hierarchy of intersections.
- Determination of the functional boundary of intersections so that an intersection of lower functional classification is not located within the functional boundary of an intersection of higher classification.
- Establishment of comparability between the intersections of two public streets and the intersection resulting from the connection of private access drives to public streets.
- Design of intersections (private drives as well as public streets) so that left- and right-turning vehicles do not cause serious interference with through traffic.
- Design of medians and median openings to provide access control at unsignalized intersections, public as well as private.
- Visibility to the driver of the location and geometries of each intersection

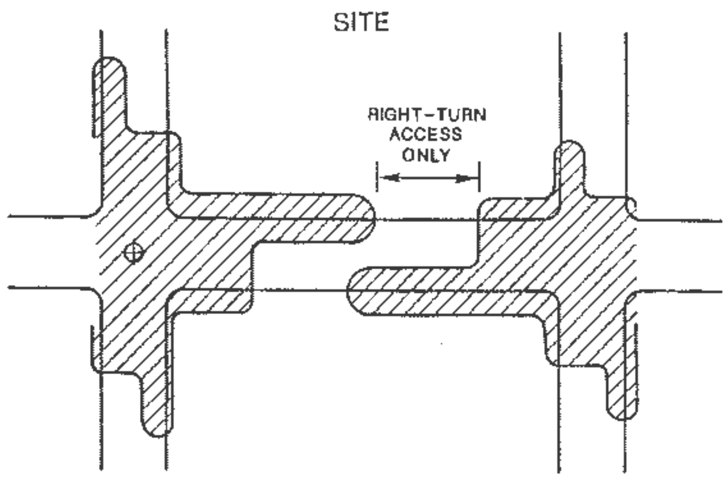
He notes that while AASHTO states that “a driveway should not be located within the functional boundary of an intersection,” it does not present guidelines as to the size of the functional area. He states that the logic indicates that it must be much larger than the physical area and that it should be composed of the distance traveled during the PRT plus the distance required to move laterally and come to a stop plus any required storage length. Figure 1 illustrates the elements of the functional area of an intersection and Figure 2 shows the region in which direct access might be permitted.

Several states have access management plans. The following is a summary of parts of plans relative to roadway design:

- Colorado uses a regulatory method for access control on state highways. Based on their experiences, Demosthenes (*19*) defines access management as “the strict control of the design and operation of all driveways and public street connections onto the highway.” He states that “access control regulations should address driveway spacing, intersection and signal spacing, the denial of access requests, access geometric design including turn lanes and related design warrants.”
- Florida, along with Colorado and New Jersey, have established the United State’s first comprehensive access management programs. Sokolow (*20*) provides guidance on practical considerations when considering the institution of a comprehensive access management program by discussing the following questions:
  - What access features will you manage?
  - How will you develop a classification system for your roadways, or should you develop one at all?



Condition Where Left- and Right-Turn Access Might Be Permitted



Condition Where Right-Turn Access Only Should Be Permitted

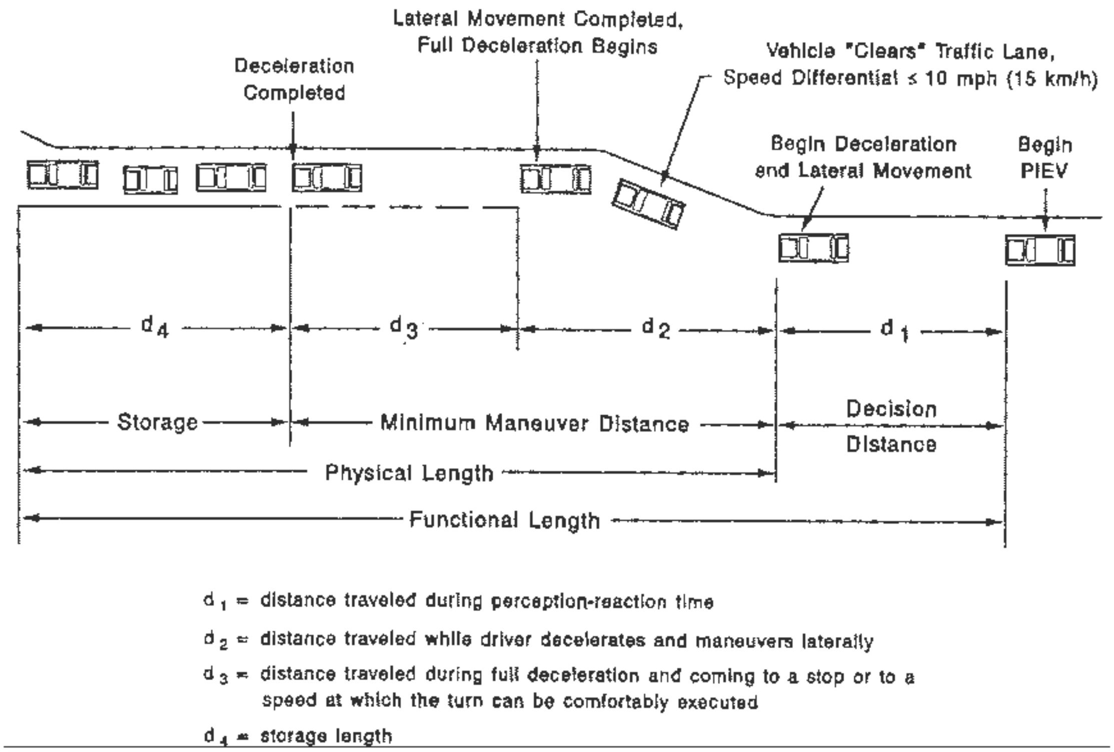
FIGURE 1 Region in which direct access might be provided on the basis of functional boundaries of the intersections adjacent to a site (*15*).

- How will you handle variances to the standards?
- How will you deal with land that has been subdivided into small lots?
- Who will administer your program?
- Should you charge fees?
- What sort of permit types will you have?
- How will you handle "grand fathering" and lane uses that redevelop?

- Poe et al. (*21*) found that access and land-use variables along with horizontal curvature influence driver's selection of operating speed on low-speed urban streets.
- Stover et al. (*22*) noted that improved capacity can be achieved on major arterial streets with the implementation of access control. Access control can be an effective method for managing congestion and is a necessary part of a congestion management system. Table 12 shows the relationship between signal spacing, speed, and cycle length. Maximum progression efficiency is achieved with 0.5 mile (0.804 km) signal spacing, cycle length of 120 sec, and a speed of 30 mph (48 km/h). Guidance on medians, left-turn bays, and right-turn bays was also provided.

Levinson et al. (*23*) also examined signal spacing in terms of its impact on performance of urban and suburban highways. Using a synthesis of relationships established in previous studies, they determined the required spacing for various speeds and cycle lengths as shown in Table 13.

Access control improves traffic operations and reduces accident experience. The FHWA synthesis on safety effectiveness of access control (*24*) stated that accident and fatality rates on facilities with full control of access to be one-half that of rural highways with no access control and



PIEV = perception, intelligence, emotion, and volition, commonly known as perception–reaction time.

FIGURE 2 Elements of the functional area of an intersection (*18*).

TABLE 12
OPTIMAL CYCLE LENGTHS FOR VARIOUS SPEEDS AND SIGNAL SPACINGS (*22*)

Speed, mph (km/h)	Optimal Cycle Lengths* (sec)		
	Signal Spacings		
	0.25 mile (0.402 km)	0.33 mile (0.536 km)	0.5 mile (0.804 km)
15 (24)	120		
20 (32)	90	120	
25 (40)	72	96	
30 (48)	60	80	120
35 (56)	51	69	103
40 (64)	45	60	89
45 (72)		53	80
50 (80)		48	72
55 (88)			65
60 (97)			60

* Maximum progression efficiency.

one-third that of urban highways of similar design. They also reported that from a series of studies in North Carolina on the relationship between median openings and accident experience, they found that traffic volume and various measures of access were the most significant contributors to accidents and that median openings should be kept to a minimum.

McGee and Hughes (*25*) reviewed safety impacts of access management in 1993. They state that research has documented that improved access management can and has produced safety benefits in terms of accident reductions. They caution, however, that the results available are based on dated information, which raises the question as to whether the estimates are still applicable. Additional research is needed to develop a more definitive relationship between safety and access management and to improve the applicability of available procedures.

In 1996, Levinson and Gluck (*26*) reported on their study of safety experience with access management. They reviewed 11 research efforts conducted between 1960 and 1980 and 11 studies conducted since the mid-1980s. Table 14 reproduces their summaries of the 11 more recent studies.

TABLE 13
OPTIMUM SIGNAL SPACING AS A FUNCTION OF SPEED AND CYCLE LENGTH (*23*)

Cycle Length (sec)	Speed, mph						
	25	30	35	40	45	50	55
60	1,100	1,320	1,540	1,760	1,980	2,200	2,420
70	1,280	1,540	1,800	2,060	2,310	2,590	2,830
80	1,470	1,760	2,030	2,350	2,640	2,940	3,230
90	1,650	1,980	2,310	2,640	2,970	3,300	3,630
100	1,840	2,200	2,570	2,940	3,300	3,670	4,040
110	2,020	2,420	2,830	3,230	3,630	4,040	4,440
120	2,200	2,640	3,080	3,520	3,960	4,400	4,840
Cycle Length (sec)	Speed, km/h						
	40	48	56	64	72	80	88
60	330	400	470	530	600	670	730
70	390	470	540	620	700	780	860
80	440	530	620	710	800	890	980
90	500	600	700	800	900	1000	1100
100	560	670	780	890	1000	1110	1220
110	610	730	860	980	1100	1220	1340
120	670	800	930	1070	1200	1330	1470

Note: Values are in feet (upper) and meters (lower).

TABLE 14
CHRONOLOGY OF ACCIDENT STUDIES RELATING TO ACCESS SPACING 1980–1990 (*26*)

Study No.	Year	Description	Findings
1	1985	Araphoe and Parker Roads, Denver [4.35 and 5.16 mi (7.0 and 8.3 km)]	Two highly access-managed roads had about 40% of the accident rate of roads with frequent access.
2	1986	Waushara County, Wisconsin	Annual accidents per mile for access spacings less than 300 ft (91.5 m) were about 2 to 3 times greater than for longer spacings.
3	1992–1993	Sokolow, Long et al., Florida	Accident rates doubled when driveways exceeded 20 per mile (12.4 per km) (Sokolow). Accident rates increased 70% as driveways per mile increased from less than 13 to more than 20 (8.1 to more than 12.4 driveways per km) (Long et al.).
4	1993	British Colombia [176 road sec- tions, 465 mi (748.7 km)]	Accident rates increased as access density increased. Each business access impacted accident rates about 50% of public road intersections.
5	1993	Millard, Lees County, Florida	Doubling connections from 20 to 40 per mile (12.4 to 24.8 per km) doubled accident rate. Doubling signals from 2 to 4 per mile (1.2 to 2.5 per km) more than doubled the accident rate.
6	1994	Michigan	Midblock accident rates generally increased as the number of intersections per mile (including driveways) and the number of lanes increased.
7	1995	Fitzpatrick and Balke (Texas)	Total and midblock accidents generally increased as driveways became more numerous.
8	1995	Lall et al., Oregon [US Route 101–29 mi (46.7 km)]	Accidents per mile and driveways per mile followed similar patterns (except for road sections with a nontransversable median).
9	1996	Norwalk–Wilton, Connecticut (Route 7)	Accident rate per mile increased along roadway carrying 20,000 to 25,000 vehicles per mile as access density increased.
10	1996	Garber and White, Virginia [10 mi (16.1 km), 30 locations]	Multiple regression analysis assessed effects of ADT/lane, average speed, number of access points, left-turn lane availability, average access spacing, and average difference in access spacing.
11	1997	Australia	Each additional driveway per kilometer increased accident rates about 1.5% for 2-lane roads and 2.5% for 4-lane roads.

Note: ADT = average daily traffic.

They also performed a comprehensive safety analysis on 37,500 accidents occurring on 264 roadway segments in 8 states. They developed accident rate indices (see Table 15) and accident rates by median type and access density from their literature synthesis and safety analysis. The relationships were adjusted to eliminate apparent anomalies in the reported data. They conclude by stating that access management does improve safety. Although the specific relationships vary, reflecting variations in road geometry, travel speeds, and driveway and intersection patterns, the

TABLE 15
SUGGESTED ACCIDENT INDICES FOR UNSIGNALIZED ACCESS SPACING (*26*)

Accident Point per Mile (accident point per kilometer) [total of both directions]	Literature Synthesis	Safety Analysis	Suggested Values
10 (6.2)	1.0	1.0	1.0
20 (12.4)	1.3	1.4	1.4
30 (18.6)	1.7	1.8	1.8
40 (24.8)	2.1	2.1	2.1
50 (31.1)	2.8	2.3	2.5
60 (37.3)	4.1	2.5	3.0
70 (43.5)	—	2.9	3.5

TABLE 16
MINIMUM AND DESIRABLE RAMP-TO-INTERSECTION SPACING FOR TWO-LANE FRONTAGE ROADS (*29*)

Exit Ramp Volume (vph)	Right Turn Percent	Exit Ramp Frontage Road Volume (vph) 1,000 Minimum ft (m)	1,000 Desirable ft (m)	2,000 Minimum ft (m)	2,000 Desirable ft (m)
250	<50	491.8 (150)	491.8 (150)	491.8 (150)	770.5 (235)
	>50	491.8 (150)	491.8 (150)	491.8 (150)	3,459.0 (1055)
750	<50	491.8 (150)	1,180.3 (360)	491.8 (150)	2,000.0 (610)
	>50	491.8 (150)	1,409.8 (430)	491.8 (150)	2,229.5 (680)
1,250	<50	491.8 (150)	2,409.8 (735)	983.6 (300)	3,229.5 (985)
	>50	491.8 (150)	2,639.3 (805)	1,229.5 (375)	3,459.0 (1055)

Note: vph = vehicles per hour.

general relationship—the greater the frequency of driveways and intersections the greater the number of accidents—remains constant.

Box (*27*) presented his views on spacing between driveways and intersections and techniques for conducting studies on driveway accidents. He studied more than 15,000 accidents in two Illinois suburbs and found that driveway accidents were related to intersections for only 1.2 percent and 2.0 percent of the total accidents. The cities do not limit driveway proximity to intersections, so, he questions the driveway spacing required in other states, which range between 125 and 660 ft (38.1 and 201.3 m). He concludes that a positive case for restrictions on driveway spacings from intersections or between driveways has not been found *based on safety* other than the values included in an ITE Recommended Practice (*28*).

Three Texas studies identified spacing needs of ramps to intersections or driveways on freeway frontage roads. For a one-way frontage road, a desirable spacing between an exit ramp followed by an entrance ramp and connected by an auxiliary lane is 984 ft (300 m) (*29*). If this length is not achievable, then the absolute minimum length should be approximately 656 ft (200 m). Minimum exit ramp-to-intersection spacings were determined for two-lane, three-lane, and two-lane with auxiliary lane frontage roads (*29*). Table 16 lists a sample of the values for the two-lane frontage road configuration.

A later Texas study evaluated the operation of frontage roads with unsignalized access located at varying distances from exit ramp terminal points (*30*). Guidelines were developed based on a combination of the distance to weave requirements and density equations formulated from computer simulation. A practical maximum of 984 ft (300 m) was placed on all configurations (see Table 17). Another Texas study developed guidelines for driveways to entrance ramps along freeway frontage roads (*31*). The analyses used consisted of operational and crash/safety assessments, both of which were based on field data specifically collected as part of the research and historical data. The results of the research indicated that an adoption of new "desirable" guidelines should be pursued to accompany the current guidelines, the latter of which it was suggested be retained as "absolute minimum" spacing guidelines. The new "desirable" guidelines serve to double the distance in existing guidelines in relation to both upstream and downstream placement of driveways in relation to entrance ramps. Therefore, the absolute minimum of 100 ft (30.5 m) changes to a desirable spacing of 200 ft (61.0 m) upstream of the ramp and the absolute minimum of 50 ft (15.3 m) to a spacing of 100 ft (30.5 m) downstream of the ramp.

PEDESTRIAN

Cheng (*32*) investigated the trend of Utah's pedestrian accident rate. The results show that Utah's accident rate for fatally injured pedestrians decreased in 1980, but held

TABLE 17
RECOMMENDED DESIRABLE SPACING GUIDELINES BETWEEN EXIT RAMP TERMINAL POINT AND THE NEAREST FRONTAGE ROAD ACCESS (*30*)

Total Volume (vph)	Driveway Volume (vph)	No. of Weaving Lanes [Spacing, ft (m)]		
		2	3	4
<2,000	All	245.9 (75)[a]	245.9 (75)[a]	245.9 (75)[a]
	<250	459.0 (140)	459.0 (140)	557.4 (170)
	>250	524.6 (160)	459.0 (140)	557.4 (170)
>2,000	>500	590.2 (180)	459.0 (140)	557.4 (170)
	>750	786.9 (240)	459.0 (140)	557.4 (170)
	>1,000	983.6 (300)	459.0 (140)	557.4 (170)
	<250	918.0 (280)	459.0 (140)	557.4 (170)
	>250	950.8 (290)	459.0 (140)	557.4 (170)
>2,500	>500	983.6 (300)	459.0 (140)	557.4 (170)
	>750	983.6 (300)	590.2 (180)	688.5 (210)
	>1,000	983.6 (300)	786.9 (240)	885.2 (270)
	<250	983.6 (300)	754.1 (230)	852.5 (260)
	>250	983.6 (300)	819.7 (250)	918.0 (280)
>3,000	>500	983.6 (300)	983.6 (300)	983.6 (300)
	>750	983.6 (300)	983.6 (300)	983.6 (300)
	>1,000	983.6 (300)	983.6 (300)	983.6 (300)

Note: vph = vehicles per hour.
[a]Absolute minimum under all conditions.

fairly constant after that with minor fluctuations. The analysis also indicates the following:

- Age 5 to 10 is the major grouping involved in Utah's pedestrian accidents.
- Most pedestrian accidents occurred during daylight, with the peak from 3:00 PM to 7:00 PM.
- The majority of pedestrian accidents are caused by pedestrian error.
- Pedestrian accidents tend to be more serious—approximately 4 percent result in fatalities.
- More pedestrian accidents occurred between intersections than at intersections.
- Males are involved in more than twice as many pedestrian accidents as females.
- Adverse road, weather, and light conditions are not a factor in most pedestrian accidents.
- Traffic control devices do not guarantee a safety zone for pedestrians.

The analysis for school-age pedestrian accidents reveals similar findings. Because most pedestrian accidents are caused by pedestrian error, more emphasis must be placed on modifying the training and behavior of pedestrians in crossing techniques, particularly for those of school age.

In 1998, the Institute of Transportation Engineers (ITE) published *Design and Safety of Pedestrian Facilities* (*33*). The report discusses guidelines for the design and safety of pedestrian facilities to provide safe and efficient opportunities for people walking near streets and highways. The report includes information on roadway design considerations, pedestrians with disabilities, sidewalks and paths, pedestrian and motorist signing, signalization, crosswalks and stop lines, pedestrian refuge islands, pedestrian barriers, curb parking restrictions, grade-separated crossings, school practices, neighborhood traffic control measures, pedestrian-oriented environments, transit stops, and work zone pedestrian safety.

Pietrucha and Opiela (*34*) examined the *Green Book* from a pedestrian design perspective to determine the adequacy of highway design standards in considering the pedestrian, the appropriateness of current design treatments, the compatibility of pedestrian and highway facility designs, and the effectiveness of the various treatments. They comment that some changes and short additions in areas such as sidewalks/walkways, refuge islands, and sidewalk flares are suggested to improve the information available to the designer. They believe that separate sections dealing with pedestrians are not necessary. Passages regarding the pedestrian's place in the movement hierarchy and the functional relationships could be woven into the existing text. Improvements to the *Green Book* that would encourage or help the highway designer "think about the pedestrian" would promote safer and more convenient designs. The following research topics dealing with the integration of pedestrian needs into highway design were suggested:

- It is necessary to investigate the functional classification scheme used in the *Green Book* to determine whether a new scheme could be devised that considers both vehicles and pedestrians.
- Increased roadway costs may result from incorporating features for pedestrians. A thorough analysis of the life-cycle costs and benefits of such actions would be useful in establishing pedestrian-sensitive design standards.
- The modifications of highway design features or the incorporation of other features can result in added

maintenance costs. There is a need to determine how these features can be designed to minimize maintenance needs and costs.
- A major difficulty in improving streets and highways to better accommodate the pedestrian is the extent of current facilities and established access patterns. The need exists to find effective concepts for the retrofitting of highways to accommodate the pedestrian.

The Public Rights-of-Way Access Advisory Committee (PROWAAC) was convened by the U.S. Architectural and Transportation Barriers Compliance Board (the Access Board) to address access to public rights-of-way for people with disabilities. Towards the end of the preparation of this synthesis, PROWAAC completed their report "Building a True Community" (*35*). This report discusses many issues including traffic calming, roundabouts, and overall design for pedestrians. It provides a new national set of guidelines that define the details necessary to make the streetscapes in public rights-of-way accessible to all users. The committee notes that ". . . the guidelines proposed do not call for a minor adjustment here and there, they ask for a dramatic change from the way public rights-of-way have been designed in the past." The report also states "It is important to understand that the recommended standards, if adopted, will apply whenever new streets are created and whenever existing streets are reconstructed or otherwise altered in ways that affect their usability by pedestrians. Implementation of these recommendations will not require jurisdictions to rebuild existing streets solely to meet these standards" (*35*). The report is available online at www.access-board.gov.

Additional reports or papers that discuss pedestrian safety are included in other chapters of this Synthesis where appropriate.

BICYCLE

The most recent and comprehensive document related to the design of facilities for bicycles is the AASHTO *Guide for the Development of Bicycle Facilities* (*36*). The *Guide* is designed to provide information on the development of facilities to enhance and encourage safe bicycle travel and to help accommodate bicycle traffic in most riding environments. It is not intended to set forth strict standards, but rather to present sound guidelines that will be valuable in attaining good design that is sensitive to the needs of both bicyclists and other highway users. Some sections of the guide include suggested minimum guidelines; however, they are only recommended where further deviation from desirable values could result in unacceptable safety compromises. Such guidelines are provided on planning; design of shared roadways, signed shared roadways, bike lanes, shared-use paths, and other design considerations; operation and maintenance; and a review of legal status.

SAFETY

During the 1990s several general safety documents were published including:

- *Accident Mitigation Guide for Congested Rural Two-Lane Highways* (*37*). This NCHRP report is designed to assist the practitioner in (1) identifying current and potential problem locations for detailed analysis, (2) isolating accident types and contributing factors, (3) matching countermeasures to accident types and contributing factors, and (4) assessing the effects of applying the countermeasures. The guide presents an accident-mitigation process, describes many countermeasures and their effects, provides examples of safety improvements, and suggests further readings.
- *Improved Safety Information to Support Highway Design* (*38*). Crash data have traditionally provided the basis for determining locations and causes of highway network safety problems. This NCHRP report presents (1) a comprehensive review of the critical safety data needs for highway design purposes; (2) an assessment of methods to gather and use data; (3) an evaluation of emerging technologies for collecting, processing, storing, and accessing data; and (4) conceptual designs for a design decision support system for safety that would be possible with improved safety information.
- *Developing Traffic Control Strategies* (*39*). This National Highway Institute course is for state and local personnel responsible for the development of traffic control plans. It develops technical skills in work zone strategies, which is the science (or technical ability) to plan and develop large highway projects that optimize the relationship between project costs, societal cost (costs to the community), highway safety, and traffic management. The curriculum includes state-of-the-art traffic control and management strategies and discussion of the advantages and disadvantages of each concerning safety and traffic management. Potential operational problems associated with specific strategies when applied to common activities are identified along with suggested mitigations. Suggested specifications and/or special provisions to contract for innovative strategies are also included.
- *Roadside Design Guide* (*40*). This AASHTO guide is a synthesis of current information and operating practices related to roadside safety. The roadside is defined as that area beyond the traveled way (driving lanes) and the shoulder (if any) of the roadway itself. Although it is a readily accepted fact that safety can

best be served by keeping motorists on the road, the focus of this guide is on safety treatments that minimize the likelihood of serious injuries when a driver does run off the road. The document is a guide rather than a standard or design policy; it is intended for use as a resource from which individual highway agencies can develop standards and policies. Issues addressed by the guide include roadside safety; economics; topography and drainage features; sign and luminaire supports; roadside and median barriers; bridge railings and transitions; barrier end treatments and crash cushions; traffic barriers, traffic control devices, and other features for work zones; and roadside safety in urban and/or restricted environments.

- *Safer Roads: A Guide to Road Safety Engineering* (*41*). This report contains a comprehensive review of the best practice approaches of road safety engineering from Europe, North America, and Australia—illuminating the practices and procedures used in the identification of hazardous sites and the development of road and traffic countermeasures. It outlines the key components of creating and maintaining a database, methods of statistical analysis, and the essential features of human behavior as they influence road and traffic design. Also covered are the economic appraisals of road safety projects and methods of project monitoring. The intended audience includes local governments; road and traffic agencies; consultants in road safety engineering, traffic engineering, or highway engineering; and students of courses in these disciplines and in road safety.
- *Road Safety Audit* (*42*). This is a guide of practice in the safety of roads and the prevention of accidents. Topics addressed include an introduction to road safety audit; the what, why, when, and who of road safety audit; road safety audit and quality assurance; legal liability issues for road authorities; a step-by-step guide to conducting an audit; six real-life case studies; road safety principles; a reading list; and a complete set of national checklists covering the five audit states from Feasibility (Stage 1) to Existing Road (Stage 5).
- *Effect of Highway Standards on Safety* (*43*). This NCHRP report reviews the literature on the relationship between highway geometric design elements and safety. The report is generally more concerned with roadway sections than with intersections, although some issues addressed in the report, such as roadside design, are potentially applicable to both.
- *Traffic Safety Toolbox: A Primer on Traffic Safety* (*44*). This ITE toolbox contains a compilation of traffic safety information on a variety of subjects providing ideas and concepts for effective traffic safety improvements. The information represents the personal knowledge, experience, and expertise of the chapter author. Chapters include safety management; traffic planning; traffic control devices—overview, signs, markings, signals, delineation; tort liability, risk management, and sign inventory systems; geometry design—cross section and alignment, sight distance, urban intersections, access-controlled facilities; one-way streets and reversible lanes; roadside safety; infrastructure maintenance—pavements, bridges, traffic control devices; work zone traffic management; designing for pedestrians; bicycling element; enforcement; driver behavior and qualification; traffic calming; and teaching safety.

The FHWA is developing the Interactive Highway Safety Design Model (IHSDM) in an attempt to marshal available knowledge about safety into a more useful form for highway planners and designers (*45*). IHSDM will be a series of evaluation tools for assessing the safety impacts of geometric design decisions. IHSDM's evaluation capabilities will help planners and designers maximize the safety benefits of highway projects within the constraints of cost, environmental, and other considerations. A small increase in the safety cost-effectiveness of individual highway projects, when accumulated across the tens of billions of dollars invested in highway improvements each year, can contribute significantly to FHWA's strategic safety objective to reduce the number of highway-related fatalities and serious injuries by 20 percent in 10 years. The development of IHSDM is a long-term, multiyear activity. The initial development efforts are restricted to two-lane rural highways—the largest single class of highways in the United States, representing approximately two-thirds of all federal-aid highways. Release of the full model for two-lane rural highways is scheduled for 2002. A subsequent phase of IHSDM development will add the capability to evaluate multilane design alternatives.

IHSDM consists of several analysis modules.

- Crash Prediction Module—To estimate the number and severity of crashes on specified roadway segments.
- Design Consistency Module—To provide information on the extent to which a roadway design conforms with driver expectations. The primary mechanism for assessing design consistency is a speed-profile model that estimates 85th percentile speeds at each point along a roadway. Potential consistency problems for which alignment elements will be flagged include large differences between the assumed design speed and the estimated 85th percentile speed, and large changes in 85th percentile speed between successive alignment elements.
- Driver/Vehicle Module—Will consist of a Driver Performance Model linked to a Vehicle Dynamics Model. The Driver Performance Model will estimate drivers' speed and path choice along a roadway.

These speed and path estimates will be input to the Vehicle Dynamics Model, which will estimate measures including lateral acceleration, friction demand, and rolling moment. Conditions that could result in loss of vehicle control (i.e., skidding or rollover) will be identified.

- Intersection Diagnostic Review Module—Will use an expert system approach to evaluate intersection design alternatives, identify geometric elements that may impact safety, and suggest countermeasures.
- Policy Review Module—To verify compliance with highway design policies. The module will identify design elements not in compliance with policy and explain the policy violated. In response to this information, the user may either correct any deficiency or prepare a design exception. To aid in evaluating the safety implications of these alternatives, the module will prompt the user to conduct further analyses with other IHSDM modules.
- Traffic Analysis Module—Will use traffic simulation models to estimate the operational effects of road designs under current and projected traffic flows. The Traffic Analysis Module will provide information on travel time, delay, interaction effects between vehicles, traffic conflicts, and other surrogate safety measures.

DESIGN CONSISTENCY

The design consistency-related work in the 1990s can be divided into four areas: work done early in the decade by Lamm and others, work done in other countries, work done in association with a 1995 FHWA study, and work done in association with a 1999 FHWA study.

Lamm and Others

Lamm et al. (*46*) reviewed design guidelines to identify controls on maximum and minimum lengths of tangents between successive curves. Minimum tangent lengths are prescribed to promote operating speed consistency, and maximum lengths are suggested to combat driver fatigue. The authors note that U.S. practice does not set maximum or minimum lengths of tangents; instead, current AASHTO policy favors long tangent sections for passing purposes on two-lane, rural roads. The Federal Republic of Germany states that the maximum length in meters of tangent sections between two curves may not exceed 20 times the design speed of that roadway. Minimum tangent lengths must be at least six times the design speed. France recommends that tangent sections be limited to a maximum of 40 to 60 percent of long roadway sections, with maximum single tangent lengths between 6,557 and 9,836 ft (2000 and 3000 m) so as to avoid driver fatigue.

Swiss highway officials also limit tangent lengths to limit driver fatigue. Designs that permit more than 1 minute of driving on a straight section are not permitted. AASHTO, however, supports the application of long tangents for passing, as demonstrated in the following quote: “Although the aesthetic qualities of curving alignment are important, passing necessitates long tangents on two-lane highways with passing sight distance on as great a percentage of the length of highway as feasible.”

Lamm et al. (*47*) developed a procedure for measuring the consistency of horizontal design as defined by operating speed and accident expected. Operating speeds and accident rates can be predicted for various lane widths based on the degree of curve and posted recommended speeds, as derived from measurements of 261 sites in New York State [also discussed in Lamm and Choueiri (*48*)]. They present guidelines for changes in operating speeds and acceptable accident rates for good, fair, and poor designs.

Work Done in Other Countries

Polus and Dagan (*49*) investigated several models for evaluating highway alignment consistency. The models developed were divided into three categories: geometric models, based on overall measures of alignment such as the ratio between minimum and maximum radii or relative length of curves; spectral models, based on time-series spectral analysis (a mathematical model that is often used to describe cyclical physical phenomena, such as the oscillations of sea waves or of electrical signals) of the highway alignments; and compound models, consisting of both spectral and geometric parameters. The validity of all three types of models was established from an analysis of a sample of 23 theoretical roads selected from those composed for the study and divided into six groups. The following conclusions were made:

- The spectral model is valid for describing the amount of consistency in highway design. This model has the highest correlation with logical ratings established from previous research and engineering judgment.
- The geometric model may also describe the consistency of a given design. It may be adopted whenever computing facilities required by the spectral model are not readily available.
- The compound model may also be adopted for the purpose of evaluating the amount of consistency of an alignment.

Al-Masaeid et al. (*50*) developed guidelines for evaluating the consistency of the horizontal alignment of two-way two-lane highways. They used data collected on four primary rural roads in Jordan to assist with the development of models. The authors note that the literature states

TABLE 18
MAXIMUM DEGREE OF HORIZONTAL CURVE THAT WOULD GUARANTEE CONSISTENT DESIGN FOR DIFFERENT PAVEMENT CONDITIONS AND GRADIENTS (*50*)

Pavement Condition	Gradient (%)	Maximum Degree of Curve (degree)
Good or Very Good (PSR ≥ 3)	2	5.71
	4	5.06
	6	4.00
	8	2.51
Fair or Poor (PSR < 3)	2	2.74
	4	2.11
	6	1.40
	8	0.00

Note: PSR = Present Serviceability Rating.

TABLE 19
MAXIMUM DEGREE OF HORIZONTAL CURVE THAT WOULD GUARANTEE CONSISTENT DESIGN FOR DIFFERENT PAVEMENT CONDITIONS AND LENGTH OF VERTICAL CURVE (*50*)

Pavement Condition	Length of Vertical Curve ft (m)	Maximum Degree of Curve (degree)
Good or Very Good (PSR ≥ 3)	1,049.2 (320)	2.85
	786.9 (240)	4.00
	524.6 (160)	4.86
	262.3 (80)	5.38
Fair or Poor (PSR < 3)	1,049.2 (320)	0.23
	786.9 (240)	1.43
	524.6 (160)	2.23
	262.3 (80)	2.69

Note: PSR = Present Serviceability Rating.

TABLE 20
LIMITS OF HORIZONTAL CURVE RADII THAT WOULD GUARANTEE CONSISTENT DESIGN ON CONTINUOUS CURVE (*50*)

Radius of the First Curve, m (ft)	Passenger Cars Radius of the Second Curve, m (ft)		All Vehicles Radius of the Second Curve, m (ft)	
	Maximum	Minimum	Maximum	Minimum
100 (327.9)	122 (400.0)	85 (278.7)	124 (393.4)	84 (275.4)
200 (655.7)	309 (1,013.1)	148 (485.2)	330 (1,082.0)	144 (472.1)
300 (983.6)	630 (2,065.6)	197 (645.9)	732 (2,400.0)	189 (619.7)
400 (1,311.5)	1352 (4,432.8)	236 (773.8)	1880 (6,163.9)	224 (734.4)
500 (1,639.3)	4018 (13,173.8)	267 (875.4)	NL*	252 (826.2)
600 (1,967.2)	NL	283 (927.9)	NL	275 (901.6)
700 (2,295.1)	NL	315 (1,032.8)	NL	294 (963.9)
800 (2,623.0)	NL	334 (1,095.1)	NL	311 (1,019.7)
900 (2,950.8)	NL	350 (1,147.5)	NL	325 (1,065.6)
1000 (3,278.7)	NL	360 (1.180.3)	NL	337 (1,104.9)

Note: NL = No maximum limit (straight).

that good consistent design can be achieved if the speed reduction is less than 6.2 mph (10 km/h), which can be achieved if the degree of curve is less than 4.25 degrees. Table 18 presents the maximum degree of curve for different pavement conditions and gradient. For example, if the pavement condition is good or better and the gradient is 6 percent, then a good consistent design would be achieved if the degree of simple curve is less than or equal to 4 degrees [radius = 1,409.8 ft (430 m)]. Table 19 presents the maximum degree of curve for different pavement conditions and length of vertical curve. For example, if the pavement condition is good or better and the length of vertical curve is 786.9 ft (240 m), then the maximum degree of curve should be limited to 4 degrees [radius = 1,409.8 ft (430 m)] to achieve a good design. For continuous horizontal curves, similar guidelines can be developed using curve radii instead of degree of curves. Table 20 presents the limit of horizontal curve radii that would guarantee a good consistent design. For example, if the radius of the first curve is 983.6 ft (300 m), the second curve should have a radius in the range of 619.7 to 2,400 ft (189 to 732 m) to achieve a good design for all vehicles. However, if the radius of the first curve is 1,639.3 ft (500 m), then the maximum radius of the second curve is 826.2 ft (252 m) and no maximum limit (straight section) could be used.

1995 FHWA Study

In 1995, Krammes et al. (*51*) reported on a major FHWA study that reviewed the state of the practice in highway geometric design consistency. The research investigated operating speed and driver work-load consistency evaluations of rural two-lane highway horizontal alignment. The operating-speed model was calibrated based on speed and geometry data for 138 horizontal curves and 78 approach tangents in 5 states. The driver work-load model was calibrated based on 2 occluded vision test studies on a total of 55 subjects. The operating-speed data suggest that 85th percentile speed generally exceeds the design speed of the horizontal curve, where design speed is less than drivers' desired speed (i.e., 85th percentile speed on long tangents). An evaluation of the relationship between speed reduction and accident experience showed that accident experience increases as the amount of speed reduction between an approach tangent to a horizontal curve increases. Several other papers and reports document additional efforts related to the data collected in the study (*52–57*).

1999 FHWA Study

The most recent major work on design consistency was a 1999 FHWA study that was to expand the research conducted in the 1995 FHWA study (*51*) in two directions: (1) to expand the speed-profile model (*58*) and (2) to investigate three promising design consistency rating methods (*59*). In addition, the research was to identify the relationship between accident frequency and the proposed design consistency methods. Following is a summary of the efforts from the FHWA research project.

Speed-Profile Model

Several different studies were undertaken to predict operating speed for different conditions, such as horizontal curves, vertical curves, and combinations of horizontal and vertical curves; tangent sections; and before or after horizontal curves. Speed data were collected at over 200 two-lane rural highway sites for use in the project. Regression equations were developed for 85th percentile, free-flow passenger vehicle speeds for the different combinations of horizontal and vertical alignment. Additionally, acceleration and deceleration rates were developed to consider the effects of horizontal curve radius.

For passenger vehicles, the best forms of the independent variable in the regression equations were $1/R$ for horizontal curves and $1/K$ for vertical curves. An example of the collected data and the developed regression equations for horizontal curves on grades is shown in Figure 3. Operating speeds on horizontal curves are very similar to speeds on long tangents when the radius is approximately 2,623 ft (800 m) or more. When this condition occurs, the grade of the section controls and the contribution of the horizontal radius is negligible. Operating speeds on horizontal curves drop sharply when the radius is less than 820 ft (250 m).

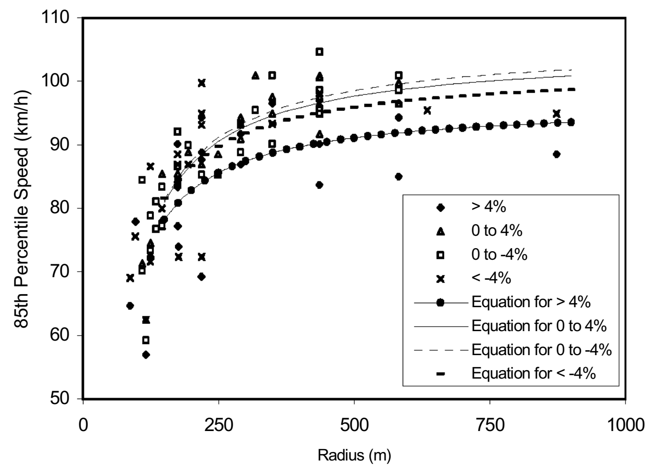


FIGURE 3 Horizontal curves on grades: V_{85} versus R (*58*).

Using the regression equations and other data, a speed-profile model was developed. The model can be used to evaluate the design consistency of a facility or to generate a speed profile along an alignment. The steps to follow in the model are shown in Figure 4. The initial step is to select the desired V_{85} speed along the roadway. Based on the findings from the research, the average 85th percentile speeds on long tangents range between 57.8 and 64.6 mph (93 and 104 km/h) for the states in this study. Therefore, a speed of 62.1 mph (100 km/h) is a good estimate of the desired speed along a two-lane rural roadway when seeking a representative, rounded speed.

The speeds predicted from developed speed prediction equations represent the speeds throughout the horizontal or vertical curves. The equations included in the TWOPAS model can be used to check the performance-limited speed at every point on the roadway (upgrade, downgrade, or level). If at any point the grade-limited speed is less than the tangent or curve speed predicted using the speed prediction equations or the assumed desired speed, then the grade-limited speed will govern. The speeds predicted from the three previous methods (assumed desired speed, speeds predicted using the speed prediction equations, and the speeds from the TWOPAS equations) are compared and the lowest speed selected (*265*). If a continuous speed

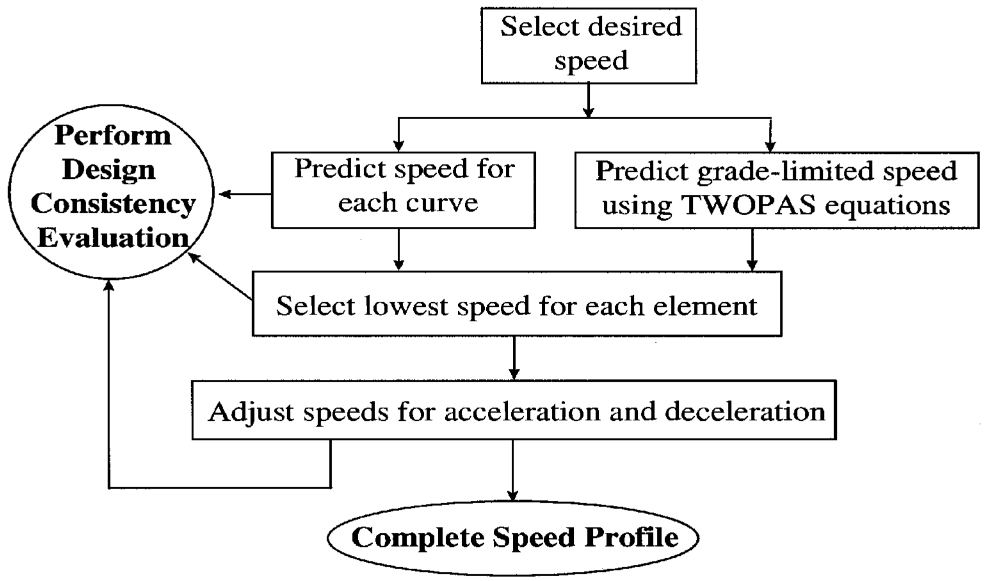


FIGURE 4 Speed-profile model flowchart (*58*).

profile for the alignment was needed, these speeds would then be adjusted for deceleration and acceleration.

The speeds for the different alignment features could be compared at any step in the speed profile model to identify unacceptable changes in speed between alignment features. For example, a flag could be raised if the speed change from one curve to another is greater than a preset value, such as 9.3 mph (15 km/h). In addition, a flag could be raised if the deceleration is greater than desired.

Speed Distribution Measures

Speed distribution measures—including variance, standard deviation, coefficient of variation, and coefficient of skewness—are logical candidates for a consistency rating method to complement speed reduction estimates from the 85th percentile speed models. The rationale for using spot speed variability measures is that inconsistent features are expected to cause more spot speed variability than consistent features and could result in more driver errors and accidents. The results from the analysis showed a low correlation between geometric alignment features and speed variance. Given this finding, it is not appropriate to consider speed variance as a design consistency measure for horizontal curvature.

Alignment Indices

Alignment indices are quantitative measures of the general character of a roadway segment's alignment. Geometric inconsistencies can arise when the general character of alignment changes between segments of a roadway. None of the alignment indices studied in this project, however, were statistically significant predictors of the desired speeds of motorists on long tangents of two-lane rural highways.

Driver Work Load/Visual Demand

A consistent roadway geometry allows a driver to accurately predict the correct path while using little visual information processing capacity, thus allowing attention or capacity to be dedicated to obstacle avoidance and navigation. A way of measuring the amount of work load or visual information needed is to use visual demand. Visual demand reflects the percentage of time that a driver is observing the roadway and is measured using a vision occlusion procedure. During the procedure drivers wore a liquid crystal display visor that was opaque except when the driver requested a 0.5-sec glimpse through the use of a floor-mounted switch.

Visual demand was determined at three types of facilities: test track environment (24 subjects driving 6 single curves and 4 paired curves for 6 runs), on-road (6 subjects driving 5 curves for 4 runs), and simulation (24 subjects driving 12 curves for 6 runs). When comparing the findings between the different studies, statistical analysis showed that no significant difference in slope (with respect to the inverse of radius) existed between all but one of the comparisons. This finding provides a level of confidence that work-load differences between features can reliably be predicted. The comparisons between intercepts, or constants, however, showed that those intercepts were generally significantly different.

The finding that there is no difference in the slope of the regression line when comparing test track results with on-road results, but that there is a difference in the intercept, would indicate that *relative* levels of work load can be ascertained, but not *absolute* levels. This finding shows promise in determining *differences* in work-load levels between successive highway features, but not baseline levels. Because most applications of driver work load are expected to be with respect to changes in level rather than in absolute terms, the general agreement with respect to the slope of the work-load measures used is very encouraging.

Relationships of Design Consistency Measures to Safety

Of the candidate design consistency measures, the speed reduction on a horizontal curve relative to the preceding curve or tangent clearly has the strongest and most sensitive relationship to accident frequency. Other candidate design consistency measures investigated were ratio of an individual curve radius to the average radius for the roadway section as a whole, average rate of vertical curvature on a roadway section, and average radius of curvature on a roadway section. Table 21 is an example of the relationship of speed reduction between successive geometric elements and accident rate. Accident frequency is not as sensitive to the alignment indices reviewed as it is to the speed reduction for individual horizontal curves. In addition, the evaluation has shown that the speed reduction on a horizontal curve is a better predictor of accident frequency than the radius of that curve. This observation makes a strong case that a design consistency methodology based on speed reduction provides a better method for improving the potential safety performance of a proposed alignment alternative than a review of horizontal curve radii alone.

SPEED PREDICTION

Fitzpatrick et al. (*58*) developed speed prediction equations for horizontal alignments, vertical alignments, and combination alignments on two-lane rural highways. Data collected in six states at 176 sites were used to develop the speed prediction equations. Regression equations were developed for passenger cars for most combinations of horizontal and vertical alignments. Some of the combinations, however, had sample sizes that were not large enough for the equations to be considered definitive.

Table 22 summarizes the regression equations and/or the value that should be assumed for the different combinations of horizontal and vertical curve combinations. For passenger vehicles, radius was the only significant independent variable in predicting 85th percentile speeds for all alignment combinations that included a horizontal curve on grade. The best form of the independent variable in regression equations is $1/R$. Operating speeds on horizontal curves are very similar to speeds on long tangents when the radius is greater than or equal to approximately 2,623.0 ft (800 m). When this condition occurs, the grade of the section may control the selection of speeds and the contribution of the horizontal radius is negligible. Operating speeds on horizontal curves decrease with respect to decreasing radius values when the radius is less than 2,623.0 ft (800 m) and drop at a faster rate when the radius is less than 1,311.5 ft (400 m).

Passenger vehicle speeds on limited-sight-distance-vertical curves on horizontal tangents could be predicted using the rate of vertical curvature as the independent variable. The best form of the independent variable in the regression equations is $1/K$. A statistically significant regression equation was not possible for crest curves where the sight distance is not limited; therefore, the desired speed for long tangents is assumed. For sag curves on horizontal tangents, the plot of the seven available data points and the regression analysis indicate that the desired speed on long tangents should be assumed. For nonlimited-sight-distance crest vertical curves in combination with horizontal curves, the lowest speed of the speeds predicted using the equations developed for horizontal curves on grades or the assumed desired speed should be used. The collected speed data for that condition were generally for large horizontal curve radii with several of the speeds being above 62.1 mph (100 km/h). Drivers may not have felt the need to reduce their speed in response to the geometry for these large radius horizontal curves.

For the horizontal curvature combined with either sag or limited-sight-distance crest vertical curves, the radius of the horizontal curve was the best predictor of speed. In summary, the research produced speed prediction equations

TABLE 21
ACCIDENT RATES AT HORIZONTAL CURVES BY DESIGN SAFETY LEVEL (*58*)

Design Safety Level*	No. of Horizontal Curves	Three-Year Accident Frequency	Exposure (million veh-km)	Accident rate (accidents/ million veh-km)
Good: $\Delta V_{85} \le 10$ km/h	4,518	1,483	3,206.06	0.46
Fair: 10 km/h < $\Delta V_{85} \le 20$ km/h	622	217	150.46	1.44
Poor: $\Delta V_{85} > 20$ km/h	147	47	17.05	2.76
Combined	5,287	1,747	3,373.57	0.52

*ΔV_{85} = difference in 85th percentile speed between successive geometric elements (km/h).

TABLE 22
SPEED PREDICTION EQUATIONS FOR PASSENGER VEHICLES (*58*)

AC Eq. No.[a]	Alignment Condition	Equation[b]	No. of Observations	R^2	MSE
1	Horizontal Curve on Grade: $-9\% \le G < -4\%$	$V_{85} = 102.10 - \frac{3077.13}{R}$	21	0.58	51.95
2	Horizontal Curve on Grade: $-4\% \le G < 0\%$	$V_{85} = 105.98 - \frac{3709.90}{R}$	25	0.76	28.46
3	Horizontal Curve on Grade: $0\% \le G < 4\%$	$V_{85} = 104.82 - \frac{3574.51}{R}$	25	0.76	24.34
4	Horizontal Curve on Grade: $4\% \le G < 9\%$	$V_{85} = 96.91 - \frac{2752.19}{R}$	23	0.53	52.54
5	Horizontal Curve Combined with Sag Vertical Curve	$V_{85} = 105.32 - \frac{3438.19}{R}$	25	0.92	10.47
6	Horizontal Curve Combined with Nonlimited-Sight-Distance Crest Vertical Curve	—[c]	13	NA	NA
7	Horizontal Curve Combined with Limited-Sight-Distance Crest Vertical Curve (i.e., $K \le 43$ m/%)	$V_{85} = 103.24 - \frac{3576.51^{d}}{R}$	22	0.74	20.06
8	Sag Vertical Curve on Horizontal Tangent	V_{85} = assumed desired speed	7	NA	NA
9	Vertical Crest Curve with Non-Limited Sight Distance (i.e., $K > 43$ m/%) on Horizontal Tangent	V_{85} = assumed desired speed	6	NA	NA
10	Vertical Crest Curve with Limited Sight Distance (i.e., $K \le 43$ m/%) on Horizontal Tangent	$V_{85} = 105.08 - \frac{149.69}{K}$	9	0.60	31.10

Notes: NA = not available; MSE = mean square error.
[a]AC Eq. No. = Alignment Condition Equation Number.
[b]Where V_{85} = 85th percentile speed of passenger cars (km/h), R = radius of curvature (m), and G = grade (%).
[c]Use lowest speed of the speeds predicted from Equations 1 or 2 (for the downgrade) and Equations 3 or 4 (for the upgrade).
[d]In addition, check the speeds predicted from Equations 1 or 2 (for the downgrade) and Equations 3 or 4 (for the upgrade) and use the lowest speed. This will ensure that the speed predicted along the combined curve will not be better than if just the horizontal curve was present (i.e., that the inclusion of a limited-sight-distance crest vertical curve result in a higher speed).

that can be used to calculate the expected speed along an alignment that includes both horizontal and vertical curvature. For some combination, engineering judgment had to be used because sample sizes were less than desirable. The research also illustrates that predicting speeds for combination alignments is not simple and requires further study. Other research has also developed equations to predict speeds on horizontal curves. Table 23 summarizes the previous equations developed.

Andjus and Maletin (*60*) collected speed data on nine horizontal curves in Yugoslavia. The curves had radii in the range of 164 to 2,459 ft (50 to 750 m). They concluded that the basic relationships between speed and radius are similar to those defined in developed countries, but the speeds are somewhat lower because of specific characteristics of Yugoslav drivers and passenger cars.

The objectives of a recent Texas DOT project (*61*) were to identify those factors that affect speed on suburban arterials and to determine the range of the influence. Analyses were conducted on data collected at 19 horizontal curve sites and 36 straight section sites. The project investigated geometric, roadside, and traffic control device variables that may affect driver behavior on suburban arterials. Regression techniques were used to determine how selected variables affect operating speed on horizontal curves and straight sections. When all variables are considered, posted speed limit was the most significant variable for both curves and straight sections. Other significant variables for curve sections were deflection angle and access density class. In another series of analyses performed without using posted speed limit, only lane width is a significant variable for straight sections, whereas median presence and roadside development were significant for curve sections. The analysis that included posted speed limit, however, produced stronger relationships between speed and significant variables than the analysis that excluded posted speed limit.

A previous Texas project (*11*) on four-lane divided suburban arterials used data collected at 14 horizontal and 10 vertical curve sites. Regression analysis indicated that the curve radius for horizontal curves and the inferred design

TABLE 23
REGRESSION EQUATIONS FOR OPERATING SPEEDS ON HORIZONTAL CURVES IN THE UNITED STATES (*58*)

Author	Equation	R^2	Sample Size (curves)	Location	Year
Voigt	$V_{85} = 99.61 - \frac{2951.37}{R} + 0.014L - 0.13I + 71.32e$	0.84			1996
Ottesen	$V_{85} = 103.64 - \frac{3400.73}{R}$	0.80	138	NY, PA, OR, TX, WA	
Krammes et al.	$V_{85} = 102.44 - \frac{2741.81}{R} + 0.012L - 0.10I$	0.82			1993
Islam et al.	$V_{85} = 103.03 - \frac{4208.76}{R} - \frac{36597.92}{R^2}$	0.98	8	UT	1994
Lamm et al.	$V_{85} = 94.39 - \frac{3189.94}{R}$	0.79	261	NY	1986
Glennon et al.	$V_{85} = 103.96 - \frac{4524.94}{R}$	0.84	56	FL, OH, IL, TX	1985
Taragin	$V_{90} = 88.87 - \frac{2554.76}{R}$	0.86	35	NY, MD, IL MN, SC	1953

Where V_{90} = 90th percentile speed on a curve (km/h), L = length of curve (m), V_{85} = 85th percentile speed on a curve (km/h), I = deflection angle (deg), R = radius of curvature (m), and e = superelevation (m/m).

TABLE 24
COMPARISON OF WHEN 85TH PERCENTILE SPEED IS APPROXIMATELY EQUAL TO INFERRED DESIGN SPEED (*61*)

Roadway Type	Point Where 85th Percentile Speed Is Approximately Equal to Inferred Design Speed, mph (km/h)[a]
Horizontal Curves	
Two-Lane Highways	55.9 (90)
Two-Lane Rural Highways	62.1 (100)
Suburban Arterials	43.5 (70)
Vertical Curves	
Two-Lane Highways	65.2 (105)
Two-Lane Rural Highways	60.2 to 70.2 (97 to 113)
Suburban Arterials	55.9[b] (90)

[a]Operating speeds are greater than the inferred design speeds at inferred design speeds less than this value, and operating speeds are less than the inferred design speeds at inferred design speeds greater than this value.
[b]Extrapolated from data. All of the sites studied had 85th percentile speeds greater than the curves' inferred design speeds. The curves' inferred design speeds ranged between 21.7 and 41.0 mph (35 and 66 km/h).

speed for vertical curves can be used to predict the 85th percentile speed on curves. The data from the tangent sections showed no relationship between approach density and 85th percentile speed for the approach density of 4.8 to 8.1 approaches per mile (3 and 5 approaches per kilometer). The project also found the point when drivers drive faster than the inferred design speed of a horizontal curve on other functional class roadways. Table 24 lists the findings.

Voigt and Krammes (*62*) evaluated the effects of superelevation on 85th percentile speeds on curves. They concluded that the superelevation rate is statistically significant, but it adds only 1 to 2 percentage points to the explanatory power (R^2) of regression equations that include radius or degree of curvature. This marginal improvement in speed estimation did not translate into a consistently better explanatory power of accident surrogate measures based on speed estimates.

The authors also evaluated four surrogate measures for accident rates on horizontal curves: radius or degree of curvature, operating speed reduction, superelevation deficiency, and implied side friction factor. All four variables were statistically significant; however, implied side friction demand was the most comprehensive measure and produced the best results. It is calculated by the following equation:

$$f_s = \frac{V_{85}^{\ 2}}{127R} - e$$

where

f_s = implied side friction factor,
V_{85} = estimated 85th percentile operating speed (km/h),
R = actual curve radius (m), and
e = actual superelevation rate (m/m).

Anderson et al. (*63*) examined the relationship to safety of the following five candidate measures of geometric design consistency for rural two-lane highway alignments:

- Speed reduction on a horizontal curve relative to the preceding tangent or curve,
- Average radius,
- Ratio of maximum radius to minimum radius,
- Average rate of vertical curvature, and
- Ratio of individual curve radius to average radius.

All of these measures were found to have a statistically significant relationship to accident frequency in the direction expected and are recommended as candidate measures for assessment of geometric design consistency.

An FHWA study on low-speed urban streets collected vehicle speeds at 21 urban locations in Pennsylvania. Sensors were placed at a control point within the corridor, at 150 ft (45.8 m) before the point of curve, at the point of curve, at the midpoint of the curve, at the point of tangent, and 150 ft (45.8 m) past the point of tangent of the curve. In their work they noted that the 85th percentile side friction demand exceeded the AASHTO recommended design values at 56 of 63 test sensor locations on the horizontal curve. They commented that a comprehensive research effort is needed to determine what defines a "comfortable" level of side friction (*64*). They also concluded that an operating speed approach to the design of urban streets might result in use of a predictive speed model. This type of model could be used for new street construction or to examine traffic engineering and planning decisions along existing streets. Improved estimation of the vehicle operating speed prior to implementation of a design will assist designers and planners in constructing streets consistent with the intended operations. A larger database is needed to achieve this goal (*65*).

The 1994 *Green Book* summarizes the results of several studies by concluding: "Many operators drive just as fast on wet pavement as they do on dry." Lamm et al. (*66*) found no statistical difference in operating speeds on wet and dry pavements. Ibrahim and Hall found that light rain affected speeds by about 1 mph (1.6 km/h) and heavy rain had an affect of 3 to 6 mph (4.8 to 9.7 km/h) (*67*). Research on day–night speed influences generally concludes that there is a statistically insignificant difference in day versus night speeds (*68,69*). Guzman (*70*) found a significant difference in free-flow speed at some locations (but not all); however, the magnitude of the difference was small [1 to 5 mph (1.6 to 8.1 km/h)].

Dixon et al. (*71*) compare the *Highway Capacity Manual* multilane highway rules-of-thumb for free-flow speed for both 55 and 65 mph (88.6 and 104.7 km/h) posted speed limit conditions. Speed data were collected at 12 rural multilane stationary count locations when the sites were posted at 55 mph (88.6 km/h) and then at 65 mph (104.7 km/h). The authors found that the higher posted speed limit generally resulted in statistically greater free-flow speeds. Comparing the speeds determined using the *HCM* rule-of-thumb with the observed speeds demonstrated that the *HCM* rule-of-thumb tended to underestimate the observed speeds for the lower speed limit while consistently overestimating the free-flow speed for the higher speed limit. The authors developed alternative rules-of-thumb for free-flow speed estimation based on the studied sites (see Table 25). The authors noted that the alternative techniques need validation. Other additional research needs include quantifying extreme variables (e.g., steep vertical grade), less than ideal characteristics (i.e., narrow lanes), and determining if the change in speed as a result of the change in speed limits is greater after additional time has passed since the changing of the speed limit signs.

WORK-LOAD RELATIONSHIP TO ACCIDENTS

Krammes and Glascock (*54*) noted that driver work load holds potential as a method for identifying and quantifying geometric inconsistencies. Work-load values quantify the criticality of individual features and the interacting effects of combinations of features along an alignment. Using a method developed by Messer et al. for an FHWA study (*72*), the driver work-load values were determined for five roads in Texas and compared to the accidents along the roads. The statistical analysis results suggest that driver

TABLE 25
FREE-FLOW SPEEDS, RULES OF THUMB (*71*)

Source	Speed Condition	Free-Flow Speed	
		Metric (km/h)	Imperial (mph)
HCM and McShane et al. (1998)	85th percentile ≥ 60 mph (96.6 km/h)	$FFS_i = 85SP - 4.8$	$FFS_i = 85SP - 3$
	Posted ≥ 55 mph (88.6 km/h)	$FFS_i = SL + 8.1$	$FFS_i = SL + 5$
Alternative rules-of-thumb	85th percentile ≥ 60 mph (96.6 km/h)	$FFS_i = 85SP - 8.7$	$FFS_i = 85SP - 5.4$
	Posted = 55 mph (88.6 km/h)	$FFS_i = SL + 10.4$	$FFS_i = SL + 6.5$
	Posted = 65 mph (104.7 km/h)	$FFS_i = SL - 0.5$	$FFS_i = SL - 0.3$

work-load values may be good predictors of accident experience on two-lane rural roads. The authors recommended a more comprehensive study. Wooldridge (*73*) also used the work-load method developed by Messer. He examined the relationship between the work-load ratings and the accident records on 19 rural two-lane highways in Texas. He concluded that roadway sections with either high work-load magnitudes or large increases in work load were associated with high accident rates when compared with accident rates on other sections on the study roadways.

CHAPTER THREE

ELEMENTS OF DESIGN

STOPPING SIGHT DISTANCE

The NCHRP Stopping Sight Distance project (*74*) provided the most recent information on issues associated with stopping sight distance (SSD). Several papers have been developed as part of the study and can provide additional information on findings (*75–79*). To calibrate the recommended model, the research included five separate but interrelated field studies—driver braking performance, driver visual capabilities, driver eye and vehicle heights, safety studies, and operating speed studies. A summary of the key findings from the Stopping Sight Distance project include the following:

- The new SSD model is based on parameters describing driver and vehicle capabilities that can be validated with field data and defended as safe driving behavior. The recommended model is

$$SSD = 0.278Vt + 0.039\ V^2/a \qquad (1)$$

where

SSD = stopping sight distance (m),
V = initial speed (km/h),
t = driver perception–brake reaction time (sec), and
a = driver deceleration (m/sec^2).

- The new model results in SSDs (Table 26), sag vertical curve lengths, and lateral clearances that are between the current minimum and desirable requirements, and crest vertical curve lengths that are shorter than current minimum requirements.
- For consistency, it is recommended that the parameters within the recommended SSD model represent common percentile values from the underlying probability distributions. Specifically, 90th (or 10th) percentile values are recommended for design, as follows:

  – One design speed and SSD,
  – Perception–brake reaction time = 2.5 sec,
  – Driver deceleration = 11.1 ft/sec^2 (3.4 m/sec^2),
  – Driver eye height = 3.54 ft (1080 mm), and
  – Object height = 23.6 in. (600 mm).

- This research and other studies documented in the literature show that many drivers exceed the inferred design speed (design speed calculated using current criteria and existing geometry) of horizontal and vertical curves. These results do not support the use of initial speeds less than the roadway's design speed for determining SSD requirements. *Initial speed* for determining SSD requirements should be a speed that encompasses the desired speed of most drivers. When a roadway's operating speed is expected to change over time, the highest anticipated operating speed should be used to determine SSD requirements.
- This research and other studies documented in the literature show that AASHTO's 2.5-sec perception–brake reaction time for SSD situations encompasses the capabilities of most drivers (including those of older drivers). The data show that 2.0 sec exceeds the 85th percentile SSD perception–brake reaction time for all drivers, and 2.5 sec exceeds the 90th

TABLE 26
RECOMMENDED STOPPING SIGHT DISTANCES FOR DESIGN (*74*)

Initial Speed mph (km/h)	Perception–Brake Reaction Time (sec)	Perception–Brake Reaction Distance ft (m)	Deceleration ft/sec^2 (m/sec^2)	Braking Distance ft (m)	Stopping Sight Distance for Design ft (m)
18.6 (30)	2.5	68.2 (20.8)	11.1 (3.4)	33.4 (10.2)	101.6 (31.0)
24.8 (40)	2.5	91.1 (27.8)	11.1 (3.4)	59.7 (18.2)	150.5 (45.9)
31.1 (50)	2.5	113.8 (34.7)	11.1 (3.4)	93.1 (28.4)	206.9 (63.1)
37.3 (60)	2.5	136.7 (41.7)	11.1 (3.4)	133.8 (40.8)	270.5 (82.5)
43.5 (70)	2.5	159.3 (48.6)	11.1 (3.4)	182.3 (55.6)	341.6 (104.2)
49.7 (80)	2.5	182.3 (55.6)	11.1 (3.4)	238.0 (72.6)	420.3 (128.2)
55.9 (90)	2.5	204.9 (62.5)	11.1 (3.4)	301.3 (91.9)	506.2 (154.4)
62.1 (100)	2.5	227.5 (69.4)	11.1 (3.4)	372.1 (113.5)	599.7 (182.9)
68.3 (110)	2.5	250.5 (76.4)	11.1 (3.4)	450.2 (137.3)	700.7 (213.7)
74.5 (120)	2.5	273.1 (83.3)	11.1 (3.4)	535.7 (163.4)	808.9 (246.7)

percentile SSD perception–brake reaction time for all drivers.

- This research and other studies documented in the literature show that most drivers choose decelerations greater than 18.4 ft/sec^2 (5.6 m/sec^2) when confronted with the need to stop for an unexpected object in the roadway. Approximately 90 percent of all drivers choose decelerations that are greater than 11.1 ft/sec^2 (3.4 m/sec^2). These decelerations are within the driver's capability to stay within their lane and maintain steering control during the braking maneuver on wet surfaces. Thus, 11.1 ft/sec^2 (3.4 m/sec^2) (a comfortable deceleration for most drivers) is recommended as the deceleration threshold for determining required SSD. Implicit in this deceleration threshold is the requirement that the vehicle braking system and pavement friction values are at least equivalent to 11.1 ft/sec^2 (3.4 m/sec^2) (0.34 *g*). Skid data show that most wet pavement surfaces on state maintained roadways exceed this threshold. Braking data show that most vehicle braking systems can exceed the skidding friction values for the pavement.
- This research and other studies documented in the literature show that more than 90 percent of all passenger car driver eye heights exceed 3.54 ft (1080 mm). This eye height encompasses an even larger proportion of the vehicle fleet when trucks and multipurpose vehicles are included in the population. Thus, 3.54 ft (1080 mm) is recommended as the driver eye height for determining required SSDs.
- Approximately 95 percent of the taillight heights and 90 percent of the headlight heights exceed 23.6 in. (600 mm). Additionally, this research showed that accidents with smaller objects are extremely rare and of low severity. Thus, 23.6 in. (600 mm) is recommended as the appropriate object height for determining required SSDs except in those locations where the probability of rocks or other debris in the roadway is high. In those locations, a shorter object height is appropriate.

PERCEPTION–REACTION TIME

A study by Lerner (*9*) investigated age and driver perception–reaction time (PRT) for sight distance design requirements. He determined the PRT of drivers to a barrel that appears to be entering the roadway in front of the driver (a set of chains prevented the barrel from actually crossing the shoulder area). The barrel emerged into view approximately 200 ft (61 m) before the vehicle; at the target speed of 40 mph (64.4 km/h), this provided a time-to-collision of about 3.4 sec. The primary findings of the experiment were:

- The mean brake PRT for the 56 subjects was 1.5 sec, with a standard deviation of 0.4 sec. The 85th percentile time was 1.9 sec. The longest time was 2.54 sec; the next longest was 2.39 sec. Therefore, the 2.5-sec AASHTO PRT value appears to adequately cover virtually all observations.
- There was not a statistically meaningful difference in mean brake PRT due to age, gender, or their interaction.
- About one-half of the subjects (51 percent) reacted by braking (8 percent) or braking and steering (43 percent), with 36 percent steering only.

HEAD-ON SIGHT DISTANCE

Gattis (*80*) noted that where two lanes of traffic moving in opposite directions operate in one lane, as happens on many residential streets, an amount of sight distance greater than that recommended by the Stopping Sight Distance project is needed. The head-on sight distances (HSDs) that he calculated are listed in Table 27. The need for HSD may exist when parking occurs on both sides of a residential street, when parking exists on one side of more narrow streets, or the presence of vegetation or other large fixed objects at the side of the curb obstructs the driver's view. When HSD is deficient on local residential streets, parking restrictions may provide a remedy along with

TABLE 27
SIGHT DISTANCE COMPARISON (*80*)

Assumed Speed mph (km/h)	PRT (sec)	Distance During PRT ft (m)	Wet Braking Distance ft (m)	Needed Total Sight Distance for Two Approaching Vehicles ft (m)	Recommended Sight Distance* ft (m)
25 (40.3)	1.2	44.0 (13.4)	54.8 (16.7)	198 (60.4)	146 (44.5)
25 (40.3)	2.5	91.7 (28.0)	54.8 (16.7)	293 (89.4)	146 (44.5)
30 (48.3)	1.2	52.8 (16.1)	85.7 (26.1)	277 (84.5)	196 (59.8)
30 (48.3)	2.5	110.0 (33.6)	85.7 (26.1)	391 (119.3)	196 (59.8)

Note: PRT = perception–-reaction time.
*Derived from Table III-1, 1984 *Green Book*.

TABLE 28
SAMPLE OF MINIMUM SIGHT DISTANCE OF COMPOUND HORIZONTAL CURVES FOR OPEN HIGHWAYS (R_1/R_2 = 1.5) (*81*)

Obstacle location	Central Angle (deg)		Obstacle ratio[a]	Radius of Sharper Arc, R_2, ft (m)								
				655.7 (200)			1,311.5 (400)			1,967.2 (600)		
	I	*J*		M^b = 32.8 (10)	65.6 (20)	98.4 (30)	M^b = 32.8 (10)	65.6 (20)	98.4 (30)	M^b = 32.8 (10)	65.6 (20)	98.4 (30)
On sharper arc	10	10	0.0	603.3 (184)	986.9 (301)	1367.2 (417)	803.3 (245)	1209.8 (369)	1603.3 (489)	967.2 (295)	1416.4 (432)	1816.4 (554)
	10	10	0.5	537.7 (164)	924.6 (282)	1209.8 (396)	685.2 (209)	1079.1 (329)	1462.3 (446)	806.6 (246)	1232.8 (376)	1619.7 (494)
	30	20	0.0	547.5 (167)	800.0 (244)	996.7 (304)	760.7 (232)	1108.2 (338)	1377.0 (420)	941.0 (287)	1331.1 (406)	1655.7 (505)
	30	20	0.5	449.2 (137)	691.8 (211)	885.2 (270)	600.0 (183)	901.6 (275)	1157.4 (353)	724.6 (221)	1065.6 (325)	1360.7 (415)
On flatter arc	10	10	0.0	495.1 (151)	760.7 (232)	1016.4 (310)	662.3 (202)	993.4 (303)	993.4 (303)	1265.6 (386)	1180.3 (360)	1498.4 (457)
	10	10	0.5	508.2 (155)	763.9 (233)	1019.7 (311)	727.9 (222)	1016.4 (310)	1016.4 (310)	1275.4 (389)	1252.5 (382)	1527.9 (466)
	10	10	1.0	649.2 (198)	927.9 (283)	1190.2 (363)	931.1 (284)	1301.6 (397)	1301.6 (397)	1586.9 (484)	1619.7 (494)	1957.4 (597)
	20	20	0.0	452.5 (138)	665.6 (203)	849.2 (259)	645.9 (197)	908.2 (277)	908.2 (277)	1137.7 (347)	1121.3 (342)	1370.5 (418)
	20	20	0.5	511.5 (156)	721.3 (220)	885.2 (270)	731.1 (223)	1026.2 (313)	1026.2 (313)	1255.7 (383)	1259.0 (384)	1550.8 (473)
	20	20	1.0	659.0 (201)	924.6 (282)	1108.2 (338)	934.4 (285)	1324.6 (404)	1324.6 (404)	1616.4 (493)	1619.7 (494)	1983.6 (605)

[a]Obstacle ratio equals *J*1/*J* or *I*1/*I* when the obstacle lies on the sharper or flatter arc, respectively.
[b]Lateral clearance [ft (m)].
Notes: Only a sample of the minimum sight distance is presented in this table. See original reference for complete set of values. *I* and *J* = central angles of flatter and sharper arcs, respectively.

removing view-obstructing objects along the roadway. Gattis commented that more study is needed to determine the proper PRT for local residential street situations along with the proper tire friction values, the amount of sight clearance around a parked car to perceive another moving car, and the suitable assumed lateral position of the driver's eye.

COMPOUND HORIZONTAL CURVE SIGHT DISTANCE

Easa (*81*) determined the sight distance for an obstacle located within the sharper arc or the flatter arc of a compound curve. The tables he developed can be used to find the minimum sight distance on the curve and, subsequently, the lateral clearance requirements to satisfy sight distance needs. He notes that compound curves are advantageous in effecting desirable shapes of turning roadways and are more economical in mountainous terrain. For obstacles on the sharper arc, application of simple curve models that ignore the flatter arc will overestimate lateral clearance needs. For obstacles on the flatter arc, application of the simple curve models that ignore the sharper arc will underestimate the lateral clearance needs. He concludes that the design tables included in his paper should be useful in reducing the cost of providing the lateral clearance and in achieving safer operations for compound highway curves. Table 28 shows a sample of the tables that Easa developed.

HIGHWAY REVERSE CURVES SIGHT DISTANCE

Easa (*82*) investigated sight distance needs for reverse horizontal and vertical curves. He commented that simple models might overestimate the lateral clearance needs for reverse horizontal curves and the curve length requirements for reverse vertical curves. A computer program was developed to plot the sight distance profile for reverse curves in order to find the minimum sight distance along the reverse curve including the characteristics of any sight hidden zones (for horizontal curves) or sight hidden dips (for vertical curves). He found that the alignment reversal for horizontal curves improves the sight distance in comparison with that for a continuous tangent. For reverse vertical curves the alignment reversal improves the sight distance and consequently can reduce the required length of the crest arc.

HORIZONTAL CURVES

Harwood and Mason (*83*) conducted an evaluation of horizontal curve design policy. They used a sensitivity analysis to evaluate the margin of safety against vehicle skidding and rollover for both passenger cars and trucks traveling at the design speed on minimum-radius curves designed in accordance with AASHTO policy. The following conclusions and recommendations have been drawn from the results presented by Harwood and Mason concerning the

AASHTO high-speed or open-highway horizontal curve design criteria presented in the 1990 *Green Book*, Table III-6.

- On horizontal curves designed in accordance with AASHTO high-speed criteria, a passenger car with poor tires on a poor wet pavement will generally skid at a lower speed than it will roll over. However, even minimum-radius curves designed in accordance with AASHTO policy provide an adequate margin of safety against both vehicle skidding and rollover for passenger cars traveling at the design speed.
- On minimum-radius curves designed in accordance with AASHTO high-speed criteria, the most unstable trucks (i.e., those with the highest centers of gravity) will role over before they will skid off the road on a dry pavement. However, on a poor wet pavement, a truck with poor tires on a minimum-radius curve will generally skid at a lower speed than it will roll over on curves with design speeds of up to 40 mph (64 km/h); the most unstable trucks will roll over at a lower speed than they will skid off the road.
- The margins of safety against skidding and rollover by trucks appear to be adequate for trucks that do not exceed the design speed on curves designed in accordance with the *Green Book*, Table III-6.
- Variations in the methods for developing superelevation on horizontal curves, such as the provision of spiral transitions, have only very small effects on the likelihood of skidding or rolling over by trucks.
- On horizontal curves with lower design speeds that are designed in accordance with *Green Book*, Table III-6, the most unstable trucks can roll over when traveling as little as 5 to 10 mph (8 to 16 km/h) above the design speed. This is a particular concern on freeway ramps, many of which have unrealistically low design speeds in comparison with the design speed of the mainline roadway. A recent paper by Harwood and Mason reviews the existing AASHTO criteria for selecting the design speed of a ramp as it relates to the highway design speeds. The selection of realistic design speeds is critical to safety, particularly for trucks.
- On the basis of these evaluation results there does not appear to be a need to modify existing criteria for determining the radii and superelevations of horizontal curves in *Green Book*, Table III-6. Existing design policies provide adequate margins of safety against skidding and rollover by both passenger cars and trucks as long as the design speed of the curve is selected realistically. Special care should be taken for curves with design speeds of 30 mph (48 km/h) or less to ensure that the selected design speed will not be exceeded, particularly by trucks. Design of superelevation transitions according to the two-thirds/one-third rule provides an acceptable design, although spiral transitions would provide marginally lower lateral accelerations.

AASHTO policy permits the low-speed design criteria presented in the 1990 *Green Book*, Table III-17, to be used for horizontal curves at intersections and turning roadways with design speeds of 40 mph (64 km/h) or less. The following conclusions and recommendations were drawn from the evaluation of these low-speed design criteria.

- Minimum-radius horizontal curves designed in accordance with the low-speed criteria in *Green Book*, Table III-17, generally provide adequate margins of safety against skidding and rollover for passenger cars traveling at the design speed.
- For design speeds of 10 to 20 mph (16 to 32 km/h), minimum-radius horizontal curves may not provide adequate margins of safety for trucks with poor tires on a poor wet pavement or for trucks with low rollover thresholds. Revision of the criteria in *Green Book*, Table III-17, should be considered, especially for locations with substantial truck volumes. This same concern is applicable to the horizontal curve design criteria for low-speed urban streets based on *Green Book*, Table III-6.
- The *Green Book* should be revised to state explicitly that minimum radii smaller than those shown in Table III-17 should not be used, even when they appear justified by above-minimum superelevation rates.

Using computer simulation, Blue and Kulakowski (*84*) investigated the roll performance of tractor–semitrailer trucks on horizontal curves with three different types of transitions. Semitrailer data were used because the semitrailer has a higher center of gravity than the tractor, and will roll first; therefore, it is viewed as the more critical unit. The three types of transition are: (1) two-thirds of the maximum superelevation is developed before the start of the curve, (2) superelevation is fully developed at the start of the curve, and (3) superelevation is developed in a short spiral section. The results of the computer simulation showed that the spiral design is superior because it provides a more gradual transition into the curve resulting in smoother changes in lateral acceleration and roll angle and less need for driver correction when the truck is entering the curve. Development of full superelevation on the tangent did not improve roll stability over the standard two-thirds/one-third type, and actually seemed to result in slightly worse roll stability performance. This design type would not be recommended because more driver correction would be required to keep a truck on a straight road with high superelevation and there is no apparent benefit. The critical speeds (i.e., the maximum speed at which the truck will not roll over) were determined from the simulation data. In all cases, the critical speed exceeded the design speed. The safety margin ranged from 11.8 to 33.1 mph (19.0 to 53.3 km/h), and was higher for the higher design speeds.

Felipe and Navin (*85*) reported on an experiment that measured speed and lateral acceleration for drivers on four horizontal curves at a test track and then four horizontal curves on a roadway. The experimental results from the test track indicated that pavement condition (wet or dry) did not significantly affect the selected driver speed. The following were very influential on selected speed: radius and whether the driver was following the instructions of driving at a "comfortable speed" or at a "very difficult speed." The experimental observations corresponded reasonably well to the field observations. In general, drivers limited their speed on small radius curves, based on their comfortable lateral acceleration. On large radius curves, their speed was limited by both their comfortable lateral acceleration and speed environment. (Speed environment was defined as the speed a subject would travel on a straight section of the same environment, also called desired speed by others.) On small curves, a comfortable lateral acceleration was 0.35 to 0.40 *g*.

SUPERELEVATION

Several authors (*86–89*) have recently reviewed superelevation and presented suggestions on additional research needs and proposed methods for improving it. An NCHRP project report (*90*) documented the findings from a comprehensive analysis of superelevation. The research started with a literature review and survey of domestic and international practice. Data were collected at 55 curves in 8 states to quantify the relationship among side friction demand, speed, curve radius, and superelevation rate. Simulation was also used to evaluate the effects of alternative transition designs on vehicle lane position and control. Detailed recommendations for the *Green Book* were also developed in order to simplify the design of curves and to result in more consistent curve design throughout the United States. The following lists present the conclusions reached as a result of the research and the recommendations of the NCHRP Superelevation Project. The final report (*90*) provides additional details and recommended text for a future edition of the *Green Book*.

- Drivers slow on sharp horizontal curves. The magnitude of their speed reduction reflects a compromise between a desire for a comfortable level of lateral acceleration and a desire to minimize travel time. From a curve design standpoint, designers should avoid curves that are so sharp that they promote a significant speed reduction [more than 9.3 mph (15 km/h)]. However, for the design of minimum radius curves, a nominal speed reduction of 1.9 to 3.1 mph (3 to 5 km/h) was found to provide an acceptable compromise between driver comfort and travel time.
- Drivers have similar side friction demands when traveling on street and highway curves. Thus, the use of separate side friction factors for the design of curves on low- and high-speed urban streets does not appear justified.
- Significant roadway downgrade depletes the friction supply available for cornering. This depletion results from the use of a portion of the friction supply to provide the necessary braking force required to maintain speed on the downgrade. The reduction in side friction supply reduces the margin of safety for vehicles traveling on downgrade horizontal curves. The reduction in margin of safety is particularly significant for heavy trucks because of their greater weight and higher peak side friction demands.
- Superelevation Distribution Method 5, in combination with the use of multiple maximum superelevation rates, does not promote design consistency. Method 5 can yield different superelevation rates for the same speed and radius depending on the designer's choice of maximum superelevation rate.
- A kinematic analysis of a vehicle's lateral motion within the transition section indicates that proper design of this section can minimize or eliminate lateral shift. This shift manifests itself as a "drift" within the traffic lane; however, it is actually the result of unbalanced lateral accelerations acting on the vehicle as it travels through the transition. An outward shift is particularly troublesome because it requires a corrective steering action by the driver that precipitates a "critical" path radius that is sharper than that of the curve. A critical radius is associated with a peak side friction demand exceeding that intended by the designer.
- For tangent-to-curve transition designs, many agencies are not maintaining a minimum superelevation runoff length equal to 2.0-sec travel time at the design speed. Rather, these agencies are using controls that dictate runoff length on the basis of a maximum relative gradient or a maximum rate of pavement rotation. This finding and the results from a kinematic analysis of vehicle motion indicate that adherence to the "travel time" control is not essential in tangent-to-curve transition design because it does not appear to improve motorist comfort or safety.
- The *Green Book* does not explicitly address the topic of road surface drainage in the transition section. The warping of the roadway in this section can pose several drainage problems. This warping can result in there being inadequate longitudinal or lateral slope for drainage purposes, which can result in a significant reduction in the friction supply. Inadequate drainage in the transition section is particularly hazardous because additional friction demands are

placed on the tire–pavement interface during curve entry.

- A review of the literature on the safety and operational benefits of spiral curve transitions indicates that these benefits are small and can only be realized under certain limited conditions. These marginal benefits are likely to be one reason so many state DOTs (estimated to exceed 70 percent) do not require the use of spirals.
- There is evidence that spiral curve transition length has an effect on curve operations and safety. Several international agencies have adopted controls that define both a maximum and a minimum spiral length. Excessively long spirals mislead drivers about the sharpness of the impending curve. Excessively short spirals result in relatively large levels of peak lateral acceleration. A kinematic analysis of vehicle motion indicates that lateral shift in the lane can be minimized when the spiral length is equal to the driver's steering time.

The following are recommendations from the NCHRP Superelevation Project (*90*):

- Curve Design Speed—The term "curve design speed" is recommended for use in horizontal curve design. This term is defined as the expected 95th percentile speed of freely flowing passenger cars on the curve. For design applications, curve design speed is equal to the 95th percentile approach speed less the selected curve speed reduction. This speed reduction ranges from 0 km/h for the flattest curves to 5 km/h for the sharpest curves.
- Maximum Design Side Friction Factors—It is recommended that a single set of side friction factors be used for all facility types. The recommended factors represent the 95th percentile side friction demand based on an acceptable speed reduction of 1.9 to 3.1 (3–5 km/h). These factors yield minimum radii that are very similar to those currently recommended in the *Green Book*.
- Minimum Radius with Normal Cross Slope—A simpler and more direct means of determining the minimum radius with normal cross slope is recommended; the resulting radii are very similar to those currently recommended in the *Green Book.* This radius is defined using a limiting level of side friction for the outside traffic lane, relative to the curve direction.
- Superelevation Distribution for Rural Highways and High-Speed Urban Streets—To achieve consistency in curve design, a superelevation distribution method is recommended that provides a unique relationship among design speed, radius, and superelevation rate. This distribution accommodates all of the current maximum superelevation rates used by state DOTS. It yields design superelevation rates that are similar to those currently recommended in the *Green Book,* especially for maximum rates in the range of 6 to 10 percent. The recommended distribution simplifies the presentation of the design superelevation rates by reducing the number of tables to two (there are currently five tables) and reconfiguring them to provide a range of radii for selected superelevation rates.
- Superelevation Transition Design—It is recommended that the superelevation runoff and tangent runout design controls provided in Chapter 3 of *NCHRP Report 439* (*90*) be used for all facility types. These controls are applicable to low- and high-speed facilities in urban and rural areas. The main benefit derived from implementation of this recommendation is consistency in design.
- Minimum Length of Superelevation Runoff—It is recommended that the minimum length of runoff for the tangent-to-curve transition be based solely on the maximum relative gradient control. In this regard, it is recommended that adherence to a minimum length equal to 2.0 sec travel time be eliminated. This deletion will yield shorter runoff lengths when the superelevation rate is low or the design speed is high. This change will improve pavement drainage and produce a smooth pavement edge without compromising safety or operations.
- Portion of Runoff Located Prior to the Curve—The kinematic analysis of lateral motion indicated that the portion of the superelevation runoff located prior to the curve could influence the magnitude of lateral shift within the lane. The portion that minimizes this shift varies from 0.70 to 0.90 (i.e., 70 to 90 percent) and depends on speed and the number of lanes in the transition section. It is recommended that this control be specified for each alignment and consistently used on each curve of the alignment, but that its value be selected at the onset of the project based on the design speed and number of lanes in the cross section.
- Limiting Superelevation Rates—The kinematic analysis of lateral motion indicated that larger superelevation rates are sometimes associated with excessive lateral shift. Specifically, rates in excess of 8, 10, 11, and 11 percent for the 95th percentile approach speeds of 18.6, 24.8, 31.1, and 37.3 mph (30, 40, 50, and 60 km/h), respectively, and are likely associated with shifts in excess of 3.3 ft (1.0 m). The magnitude of shift for speeds of 43.5 mph (70 km/h) and above are not likely to be excessive provided that the superelevation rate is 12 percent or less. It is recommended that these limiting rates be included in the *Green Book* with the instruction that they not be exceeded without some consideration given to widening the width of the traveled way.
- Minimum Transition Grades—It is recommended the *Green Book* provide guidance on the relationship be-

tween grade in the transition section and pavement drainage. Preliminary guidance is provided in Chapter 3 of *NCHRP Report 439* (*90*). This guidance indicates the need for a minimum profile grade of 0.5 percent in the transition section. It also indicates the need for a minimum edge of pavement grade of 0.2 percent (0.5 percent for curbed streets).

- Spiral Curve Transition Design—It is recommended that the spiral curve transition design controls provided in Chapter 3 of *NCHRP Report 439* (*90*) be used for all the facility types. These controls are applicable to low- and high-speed facilities in urban and rural areas. The main benefit derived from implementation of this recommendation is consistency in design.
- Guidance on the Use of a Spiral Curve Transition—It is recommended that the *Green Book* continue to recognize the use of spiral curve transitions. However, it is also recommended that additional guidance be provided on the conditions where a spiral is likely to offer a tangible benefit, relative to the tangent-to-curve design. This guidance would be intended for those agencies that currently use spirals and would not be presented as a "warranting" condition.
- Maximum Radius for Use of Spiral Curve Transition—Present evidence indicates that spiral curve transitions may offer a safety benefit for the sharpest curves. In this regard, it is recommended that spirals be considered when the centripetal acceleration associated with the horizontal curve (= V_2/R) exceeds 4.3 ft/sec^2 (1.3 m/sec^2).
- Minimum, Maximum, and Desirable Length of Spiral Curve Transition—As noted previously, there is considerable evidence that spiral curve transition length can have an effect on operations and safety, Several international agencies have adopted controls that define both a maximum and a minimum spiral length. Therefore, it is recommended that the minimum, maximum, and desirable spiral curve length controls described in Chapter 3 of NCHRP Report 439 (*90*) be included in the *Green Book* to help designers select a safe and comfortable spiral length.

SAFETY

Zegeer et al. (*91,92*) determined the horizontal curve features that affect accident experience on two-lane rural roads and the types of geometric improvements on curves that will affect accident experience and to what extent. Horizontal curves are considered a significant safety problem on rural two-lane highways. Previous accident studies had indicated that curves experience a higher accident rate than do tangents; rates for curves range from 1.5 to 4 times those of similar tangents. Accident relationships were developed based on an analysis of 10,900 horizontal curves in Washington State. The key findings from the study were:

- Statistical modeling analyses revealed significantly higher curve accidents for sharper curves, narrower curve width, lack of spiral transitions, and increased superelevation deficiency. All else being equal, higher traffic volume and longer curves were also associated with significantly higher curve accidents.
- Based on the predictive models, the effects of several curve improvements on accidents were determined as follows:

  – *Curve flattening* reduces crash frequency by as much as 80 percent, depending on the central angle and amount of flattening. For example, for a central angle of 40 degrees, flattening a 30-degree curve to 10 degrees will reduce total curve accidents by 66 percent for an isolated curve, and by 62 percent for a nonisolated curve. Flattening a 10-degree curve to 5 degrees for a 30-degree central angle will reduce accidents by 48 and 32 percent, respectively, for isolated and nonisolated curves.
  – *Widening lanes* on horizontal curves is expected to reduce accidents by up to 21 percent for 4 ft (1.2 m) of lane widening [i.e., 8 ft (2.4 m) of total widening].
  – *Widening paved shoulders* can reduce accidents by as much as 33 percent for 10 ft (3.1 m) of widening (each direction).
  – *Adding unpaved shoulders* is expected to reduce accidents by up to 29 percent for 10 ft (3.1 m) of widening.
  – *Adding a spiral* to a new or existing curve will reduce total curve accidents by approximately 5 percent.

- Improving superelevation can significantly reduce curve accidents where there is a superelevation deficiency (i.e., where the actual superelevation is less than the optimal superelevation as recommended by AASHTO). An improvement of .02 in superelevation (e.g., increasing superelevation from .03 to .05 to meet AASHTO design guidelines) would be expected to yield an accident reduction of 10 to 11 percent. However, no specific accident increases were found for the small curves with a superelevation greater than the AASHTO guidelines. Thus, no support can be given to the assumption of increased accident risk on curves with slightly higher superelevation than currently recommended by AASHTO.
- During routine roadway paving, deficiencies in superelevation should always be improved. Spiral transitions were also recommended, particularly for curves with moderate to sharp curvature. Improvements of specific roadside obstacles should be strongly considered, and their feasibility should be determined for the specific curve situation on the basis of expected accident reductions and project costs. As a part of

routine 3R (resurfacing, restoration, or rehabilitation) improvements, horizontal curves should be reviewed in terms of their crash experience to determine whether geometric improvements may be needed. In such cases, the accident reduction factors developed in this study should be considered along with expected costs to determine whether such improvements are cost-effective. An informational guide has been developed to assist with the design of horizontal curves on new highway sections and with the reconstruction and upgrading of existing curves on two-lane rural roads. The guide also gives a step-by-step procedure for computing expected benefits and costs for a variety of curve improvements (*93*).

Al-Senan and Wright (*94*) related head-on accidents to geometric and traffic control features with the objective of identifying those factors that predict head-on accident proneness of a particular site on the Georgia state route system. The study was limited to two-lane rural roads carrying average daily traffic of at least 2,000 vehicles per day. The study used the discriminant analysis technique. The prediction of the proneness to a head-on accident is related to the following variables:

- Proportion of the section with pavement width of less than 24 ft (7.3 m),
- Weighted pavement width,
- Proportion of the section with shoulder width of less than 6 ft (1.8 m),
- Proportion of the section that is not level,
- Average highway speed limit of the section,
- Frequency of major access points on both sides, and
- Frequency of reverse curves with zero tangents.

Brenac (*95*) investigated the way that road design standards of different European countries account for safety aspects on two-lane roads outside of urban areas. His main conclusion was that the traditional design speed approach is insufficient and that formal complementary rules in road design standards, especially to improve compatibility between successive elements of the alignment, must be introduced. Other conclusions included:

- The conventional concept of design speed and the associated design practice do not seem sufficient to ensure consistency of the horizontal alignment and the safety of curves.
- Introducing the expected actual speeds (necessary in other respects, for example, to verify the sight-distance conditions) is positive, but not sufficient to complete the conventional approach.
- The introduction of consistency rules concerning the succession of the different elements of the horizontal alignment (radius of a curve following a straight section, compatibility of radii of two near curves) seems necessary from a safety point of view. These rules are found in some national standards. However, they are not homogeneous, and the corresponding knowledge probably is not sufficiently developed.
- The use of complex curves containing a succession of circular curves and same-direction transition curves may generate safety problems and should be avoided. Moreover, the rules for calculating the length of transition curves, which in the actual situation have a rather negative influence on the perception of the curvature and probably on safety, should be re-analyzed.

Brenac based his conclusions on the examination of general results of research on the relations between alignment characteristics, speeds, and accidents and on the analysis and comparison of design standards and methods. He notes that his conclusions should be verified using field data; for example, by comparing the accident experience on a sample of roads designed with different methods.

Fink and Krammes (*55*) examined the effects of degree of curve, approach tangent length, and approach sight distance on accident rates at horizontal curves. Their results were consistent with previous research; the relationship between accident rates and degree of curvature is clear and easy to quantify, whereas the effect of other factors is less clear and more difficult to quantify. A strong relationship ($R^2 = 0.94$) between accident rate and degree of curvature category was developed (mean accident rate = 0.05 + 0.23 × mean degree of curvature). The effects of approach tangent length and sight distance are not as clear. The results suggest that the effect of long tangent lengths becomes more pronounced on sharper curves, which is consistent with conventional wisdom and previous research and supports the benefits of evaluating speed consistency. The analysis of sight distance effects also suggest that extreme approach conditions (both short- and long-approach sight distance) may contribute to higher accident rates on sharper curves. They concluded that additional research to more clearly define critical ranges of approach tangent lengths and sight distance seems warranted.

Hauer (*96*) investigated the safety consequences of choosing the degree of horizontal curvature. He notes that although there exists much research and rich practice, the issue did not appear to have achieved either satisfactory closure or proper expression in design guidelines and practices. His analysis led to the following conclusions:

- For any given deflection angle, the design with the smaller *D* (degree of curvature) or larger *R* (radius) is always safer.
- Safety is strongly affected by the choice of *D* when the deflection angle is large. Geometric design standards (guidelines, polices) do not appear to take this into consideration. Therefore, the appropriateness of the guidance provided ought to be revisited.

TABLE 29
RANGES OF SAFETY CRITERIA FOR GOOD, FAIR, AND POOR DESIGN PRACTICES (*97*)

Criterion	Design Practices		
	Good	Fair	Poor
I	$\lvert V_{85i} - V_{85i+1}\rvert \le 10$ km/h	10 km/h $< \lvert V_{85i} - V_{85i+1}\rvert \le 20$ km/h	20 km/h $< \lvert V_{85i} - V_{85i+1}\rvert$
II	$\lvert V_{85} - V_d\rvert \le 10$ km/h	10 km/h $< \lvert V_{85} - V_d\rvert \le 20$ km/h	20 km/h $< \lvert V_{85} - V_d\rvert$
III	$0 \le f_{RA} - f_{RD}$	$-0.02 \le f_{RA} - f_{RD} < 0$	$f_{RA} - f_{RD} < -0.02$

Where V_{85} = operating speed (km/h),
V_d = design speed (km/h),
f_{RA} = side friction "assumed," and
f_{RD} = side friction "demand."

Notes: 10 km/h = 6.2 mph; 20 km/h = 12.4 mph.

- The concept that the safety benefit of an increase in the radius is very small when the radius is large is a misconception. The change in accidents is proportional to the change in radius length.
- A simple but approximate relationship was developed to determine the increase in the number of accidents when increasing the degree of a curve.
- A computational procedure was developed to aid in estimating the safety consequences. Hauer notes that the procedure is not perfect; however, it can be easily implemented with a spreadsheet.

Lamm et al. (*97*) developed a procedure for evaluating the horizontal alignment of two-lane rural roads on the basis of three individual safety criteria (Table 29). They note that to be effective, the safety evaluation process must be integrated into the modern highway design tools available to highway design engineers. These tools consist of computer-automated design systems for highway geometric design and normally contain a component for the design of horizontal alignment. The goal is to include safety impacts along with the "normally considered local, environmental, esthetic, and economic aspects in making decisions on a project."

VERTICAL CURVES

Wooldridge et al. (*98*) evaluated the use of zero-length and minimum-length vertical curves with respect to Texas DOT design practice, construction results, and vehicle dynamics. The evaluation included 20 zero-length vertical curves and 15 minimum-length vertical curves. They recommended the following:

- Guidelines for grade change without vertical curves: designing a sag or crest vertical point of intersection without a vertical curve is generally acceptable in the following situations:
  - Change in grade is 1.0 percent or less for design speeds equal to or less than 43.5 mph (70 km/h).
  - Change in grade is 0.5 percent or less for design speeds greater than 43.5 mph (70 km/h).
- When a grade change without vertical curve is specified, the construction process typically results in a short vertical curve being built (i.e., the actual point of intersection is "smoothed" in the field). Grade changes without vertical curves are not recommended for the following:
  - Bridges,
  - Direct-traffic culverts, and
  - Other locations that require carefully detailed grades.

Fitzpatrick et al. (*99*) used a detailed examination of crashes from a relatively large sample of limited sight distance roadways to determine if SSD was a contributing factor in crashes on roadway segments containing limited sight distance crest vertical curves. If limited sight distance is a contributing factor to crashes, it should show up in such a study. Reviewed were 439 narratives from crashes that occurred on 33 multilane and two-lane roadways with limited sight distance crest vertical curves. The findings suggest that the crash rates on rural two-lane highways with limited SSD are similar to the crash rates on all two-lane rural highways. They also suggest that the percentage of accidents involving large trucks and older drivers are similar on limited sight distance highways and all two-lane rural highways. Thus, for the range of conditions studied, limited SSD does not appear to cause a safety problem.

Fambro et al. (*100*) examined the relationship between design and operating speeds for crest vertical curves with limited sight distance. Geometric data and 3,500 paired speeds (speeds at control and crest sections) were collected at 36 sites in 3 states. The results indicated that both the 85th percentile and the mean operating speeds were well above the inferred design speeds of the crest vertical curves for the range of conditions studied, and that the lower the design speed the larger the difference between the 85th percentile speed and the design speed. The mean reductions in speed between the control and crest sections ranged between 3.1 and 1.1 mph (5 km/h and 1.8 km/h) for <40.4 mph and 49.7 to 55.9 mph (<65 km/h and 80 to 90 km/h) inferred design speeds, respectively, on two-lane highways with shoulders.

Several papers (*101–106*) examined the use of unsymmetrical vertical curves. In general, the authors state that the use of these curves can result in curves that are shorter than traditional curves, resulting in potential construction cost savings.

Cronje and Meyer (*107*) used probability density functions for speed, friction, and perception and reaction time to calculate SSD. This approach results in shorter vertical curve lengths.

Taiganidids (*108*) proposed a methodology for estimating available sight distance. Graphs can be drawn depicting the changes in available SSD, as well as in passing sight distance (PSD) along a crest vertical curve. "Crash speed" is proposed as a new method for evaluating a crest vertical curve. It is defined as the speed at which the vehicle is expected to collide with an object when the stopping maneuver may not be completed prior to reaching the object. Equations are provided for calculating available SSD, required SSD, length of SSD restriction, and crash speed. The code for a computer program was provided for calculating available SSD and crash speed values.

PASSING ON TWO-LANE HIGHWAYS

Two objectives for a paper by Mutabazi et al. (*109*) were (1) to study the safety of different locations of crossroad intersections relative to passing lanes and (2) to compare the operational efficiency of different passing lane configurations. A passing lane, for this paper, referred to added lanes on two-lane highways in level or rolling terrain. The study methods used included conducting an intersection traffic conflict field study to compare intersection locations and conducting traffic simulation to assess the best passing lane configuration using TWOPAS. The conflict field study was performed at six locations in Kansas for 8 hours each, and the TWOPAS model was calibrated using Kansas data. The recommendations from the study included the following:

- Passing lanes reduce percent time delay; however, different passing lane configurations (e.g., side-by-side, overlapping) appear to differ only marginally.
- Side road intersections are to be
  - avoided within a passing lane section, especially if high volume (defined as having left-turn main highway volume that would warrant a separate left-turn lane on a conventional two-lane section),
  - located near the middle of the passing lane if low-volume, and
  - avoided within lane drops and lane additions.

The purpose of the research performed by Polus et al. (*110*) was to develop models to quantify the major components of the passing process and to compare the results to existing highway design models and to arrive at conclusions regarding the applicability of the existing models. Data for approximately 1,500 passings were collected by videotaping 6 tangent two-lane highway sections from high observation points (to minimize any affect on driver behavior). Each section had no sight distance restrictions. The findings were compared to the 1994 AASHTO *Green Book* policy with the following conclusions:

- The overall sight distance in AASHTO is adequate or even slightly greater and therefore somewhat safer than the sight distances found in this study when a car passes another car. The AASHTO sight distance model, however, is not sufficient for a car passing a truck.
- Although the total sight distance of this study and of AASHTO coincides, its individual components are considerably different. The reaction distance and the safety margin at the completion of the passing are considerably smaller in this study than in the AASHTO policy. Conversely, the travel distances in the opposite lane are considerably longer in this study. The two differences offset each other for a total similar distance.
- The speed differential between the passing and the overtaken vehicle was not found to remain constant during the travel in the opposing lane. It decreases with an increase in the speed of the two vehicles.
- The reaction times at the beginning of the process and the safety times upon completion of the passing were not found to be sensitive to speeds, unlike the assumptions of AASHTO.
  - The AASHTO model assumes that the speed of the passing vehicle in the opposing lane is constant. This research found that almost no acceleration occurs prior to the encroachment onto the opposing lane; therefore, in accelerative passing the acceleration occurs mainly in the opposing lane.
  - A significant relationship was established between the passing distance—and therefore the needed sight distance—and the speed of the overtaken vehicle. The AASHTO model assumes a relationship with the speed of the passing vehicle.

The study found that about one-half of all passing maneuvers involved a truck being overtaken. Therefore, consideration of and adaptation for passenger cars passing trucks and buses is needed. They noted that the trends of their empirical findings regarding trucks are in accordance with the calibrated results of the theoretical models developed by Harwood and Glennon (*111*).

Sparks et al. (*112*) examined the effect of increased vehicle length on passing operations. The results of their

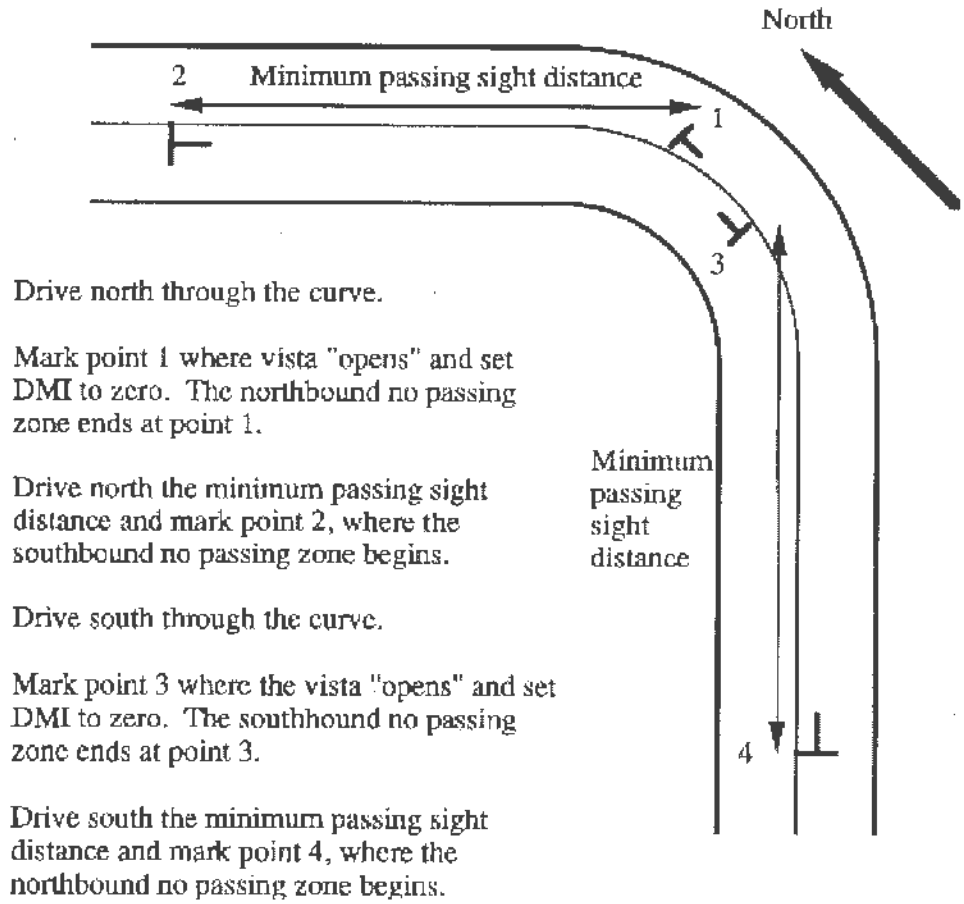


FIGURE 5 Illustration of the one-vehicle method (*113*).

modeling exercise indicated that PSDs are considerably greater when passing a long truck than when passing a car. However, whether the truck is 75.4 or 82.0 ft (23 m or 25 m) long does not have a significant impact on the PSD requirements. The differential speed between the vehicle being passed and the passing vehicle has the greatest influence in determining the PSD requirement. They also concluded that the issue of defining a critical passing maneuver to be considered in highway design (e.g., a car passing a car or a car passing a truck) needs to be resolved before deciding whether or not the current Transportation Association of Canada standard is adequate. They indicated that safety and level of service need to be considered in this decision.

Brown and Hummer (*113*) evaluated methods for measuring PSD on two-lane, two-way highways. They compared the methods at 20 horizontal curve sites and 20 hill sites, using time required to perform each method along with consideration of equipment costs, training needs, and accuracy. Based on these factors, the researchers recommended that highway agencies use the one-vehicle method. With this method, when the observer reaches the point at which the vista opens and the observer is sure there is a length of road ahead sufficient for safe passing, he or she stops the vehicle and places a paint mark on the right side of the highway. This point is the end of the no-passing zone in the direction of travel. The observer then travels the required PSD and stops to place a paint mark on the left side of the road. This marks the beginning of the no-passing zone in the opposite direction. A trip through the site in the opposite direction, following the same procedure, completes the location of the no-passing zones for the site. Figure 5 illustrates the method for a horizontal curve. The method is the same for a vertical curve.

A Canada study (*114*) on passing zones looked at the operational impacts on marking no-passing zones if the target height was reduced from the current standard of 45.3 to 23.6 in. (115 to 60 cm), which represents the standard minimum height for passenger vehicle headlights. Most vehicle manufacturers now incorporate daytime running lights into the headlight assembly. A sample of rural, two-lane arterial and collector highways in New Brunswick was selected for analysis. The measures of effectiveness were the change in percent of no-passing opportunities and the change in volume/capacity ratio. The change in target height would result in an average 8 percent increase in no-

TABLE 30
DERIVED PASSING SIGHT DISTANCE REQUIREMENTS (*115*)

Design Speed, mph (km/h)	Minimum Length of Passing Zone, ft (m)	Passing Sight Distance Requirement,[a] ft (m)
40 (64.4)	600 (183)	670 (204.4)
50 (80.5)	900 (274.5)	830 (253.2)
60 (96.6)	1,200 (366.0)	990 (302.0)
70 (112.7)	1,500 (457.5)	1,140 (347.7)

Notes: The AASHTO use of passenger cars for the passing and impeding vehicles are appropriate criteria; the length of the average passenger car is 16 ft (4.9 m); a reasonably safe deceleration rate in the abort maneuver is 8 ft/sec^2 (2.4 m/sec^2). The following critical (15th percentile) speed differentials are appropriate:

design speed, mph (km/h)		*speed differential, mph (km/h)*	
30	(48.3)	12	(19.3)
40	(64.4)	11	(17.7)
50	(80.5)	10	(16.1)
60	(96.6)	9	(14.5)
70	(112.7)	8	(12.9)

[a]Assumptions.

passing zones, thus decreasing the available passing opportunities on a facility. The average decrease in the volume/capacity ratio was found to be only 1 percent. The authors concluded that the effects on travel speeds, freedom to maneuver, and driver behavior should be minimal. They caution that the safety impact of reduced passing opportunities should be considered in order to evaluate more fully the net impacts of reducing the target-height standard. A lack of appropriate passing opportunities may result in platoon formation, driver frustration, and potential passing occurring within no-passing zones. Such adverse effects on the operational characteristics of a roadway could counteract any potential benefits derived from a modified striping standard.

Glennon (*115*) derived a mathematical model for describing the critical nature of the passing maneuver on two-lane highways. The model is based on the hypothesis that a critical position exists during the passing maneuver where the PSD requirements to either complete or abort the pass are equal. Table 30 provides the derived PSD requirements.

Glennon provided recommended passing zone lengths (Table 30, middle column) that were developed in a previous study. He commented that very short lengths, such as the 400-ft (122.0-m) default length allowed by the *Manual on Uniform Traffic Control Devices* (MUTCD), are not appropriate for safe highway operations, and that the values in Table 30 should be used unless another rationale is shown to be more appropriate. Analysis of the effect of truck length showed that the effect is not as dramatic as previously reported in the literature. Glennon observed that the PSD requirements derived with the model are considerably less than the AASHTO requirement, but are surprisingly close to those presented in the MUTCD.

Harwood et al. (*116*) presented information on effectively locating, designing, signing, and marking passing lanes to improve traffic operations. Most of the information was from a 1987 FHWA report, *Low-Cost Methods for Improving Traffic Operations on Two-Lane Highways: Informational Guide* (*117*). They also described how the operational effectiveness of passing lanes can be predicted as a function of flow rate, passing-lane length, and the percentage of traffic traveling in platoons and that the installation of a passing lane on a two-lane highway reduces the accident rate by approximately 25 percent.

TRUCK ESCAPE RAMPS

Witheford (*118*) developed a synthesis of highway practice on truck escape ramps. He found the following ramp elements to be "settled":

- The arrester bed is the preferred technique for truck escape ramps. Rounded gravel, rather than crushed aggregate, is required in at least a 36-in. (91.4-cm) bed. Uniform grading with an approximate size range of 0.5 to 0.7 in. (1.3 to 1.8 cm) provides the greatest rolling resistance and thus permits the shortest ramp lengths.
- Mounds and barrels should be used only where needed ramp length cannot be provided. Vehicles should be slowed to 25 mph (40.3 km/h) or less before reaching them.
- Beds should be straight, at a minimal angle to the roadway, and begin at a lateral distance sufficient to keep gravel from spraying back on the main roadway.
- Regulatory signing must be adequate to discourage "casual use" of ramps and stopping by other than runaway vehicles.
- Vehicle removal must be facilitated by provision of service lanes and anchor blocks.
- Maintenance must include regrading after each use and periodic "fluffing."

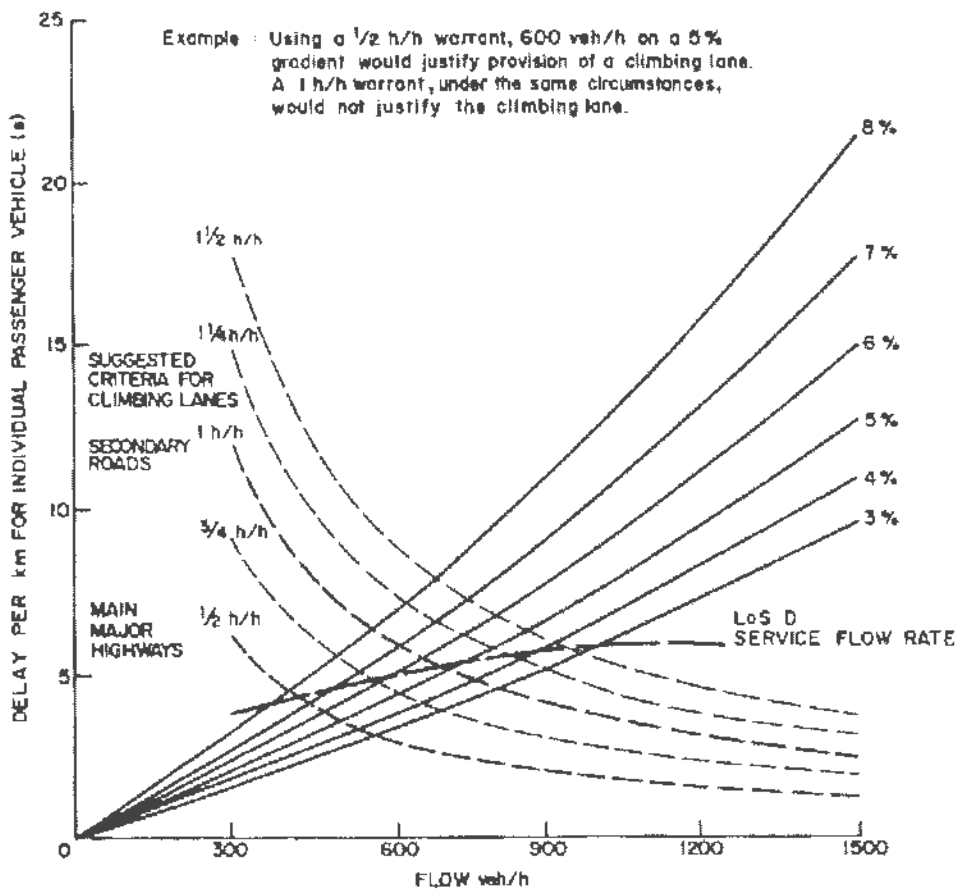


FIGURE 6 Relationship between flow, gradient, and delay (*120*).

- Provisions to avoid contamination of the bed are essential.

He also noted the following:

- Truck escape ramps are well established as a feature of the nation's highway system as confirmed by a survey of the states.
- Location and siting of truck escape ramps still pose problems, with no universally applicable answers having been found. A careful benefit–cost analysis seems the best recourse to address the question.

Basic design issues needing resolution include:

- Should entry speeds of 80 to 90 mph (128.8 to 144.9 km/h) always apply?
- Do those values combined with the current length formula provide the best answers?
- Is a width of occupancy by two vehicles justified by the frequency of multiple usage reported?

The synthesis included several proposed research topics.

CLIMBING LANES

Wolhuter and Polus (*119*) suggested that delay be used as a warrant for the installation of climbing lanes. Delay is defined as that period of time added to a trip by a reduction of space mean speed to a value less than the desired. Field data were used to calibrate TRARR, a simulation program developed by the Australian Road Research Board. The findings from the simulation were used to establish relationships between delay and flow for various gradients. Delay, suffered per kilometer by an individual passenger car, was calculated for a range of gradients (3 to 9 percent) and flows (0 to 1,500 vph). These are plotted in Figure 6. This figure also contains five lines that represent warrants for climbing lanes. These lines are based on the assumption that the total hourly delay for a given section [3,278.7 ft (1 km) in this example] should remain constant regardless of the gradient. Thus, lines for 0.5, 0.75, 1, 1.25, and 1.5 hours of total delay per hour are presented. They were generated using the following formula

$$W = Q \times D \times Pp/3600$$

where

W = constant, equal to selected warranting total delay = 0.5, 0.75, 1, 1.25, and 1.5 (h/h/km);
Q = flow (vph);
D = delay per individual passenger car(s); and
Pp = Percentage of passenger cars in stream.

A climbing lane will be justified to the right of each line and not justified to its left.

The selection of a particular criterion (0.5, 0.75, etc.) is left to the individual agency and ought to be determined beforehand based on general and economic design policies. It is suggested, for example, that on major highways, an agency may prefer a higher standard by opting for the 0.5 h/h criterion and on secondary roads accept the 1 h/h criterion.

The authors note that adopting a delay warrant over those currently in use results in a dramatic decrease in climbing lanes warranted on flatter grades and a similar increase on steeper grades. The 0.75 h/h warrant demonstrates a break-even point with the current level of service warrant at 600 pcph and a 5 percent gradient.

Homburger (*120*) investigated traffic flow at the upper end of climbing lanes on two-lane roads in California. Of 157 accidents occurring within 0.1 mi (0.16 km) of the upper end of the merging taper of 21 selected climbing lanes in a 5-year period, only 11 (7 percent) appear to have been directly caused by the need for vehicles to merge. Other circumstances, such as driving too fast for conditions, alcohol-influenced behavior, snow or ice conditions, illegal turns, and deer or rocks in the roadway, are primary factors in the majority of the accidents. Based on his analysis, he recommended the following:

- The minimum length of climbing lanes should be calculated from an estimate of platoon lengths expected to be found at the site in question.
- If the climbing lane terminates on an upgrade, the merging area should be located at a point beyond which traffic from the opposite direction can be seen for a sufficient distance to permit safe overtaking.
- Benefits obtained from a climbing lane will vary directly with the distance of the lane either from the next good overtaking opportunity or from a point where slow vehicles can resume the same speed as fast ones; that is, the summit of a grade followed by good level or downhill alignment.

COORDINATION OF HORIZONTAL AND VERTICAL ALIGNMENTS

Smith and Lamm (*121*) and Lamm and Smith (*122*) examined the coordination of horizontal and vertical alignments. In Lamm and Smith (*122*) a design process called curvilinear alignment is described, which is based on a process called relation design. Relation design means that no more single design elements with minimum or maximum limiting values are put together more or less arbitrarily; rather, design element sequences are formed in which the design elements following one another are subject to specific relations or relation ranges. The proposed curvilinear alignment design process is based on evaluating operating speed changes between successive design elements and for comparing operating speeds and design speeds of single design elements with each other. Their suggested procedures for comparing operating speed between elements and design speed to operating speed for an element are described here.

Safety Criterion I

Good Design: $|V_{85i} - V_{85i+1}| \leq 10$ km/h
Fair Design: 10 km/h $< |V_{85i} - V_{85i+1}| \leq 20$ km/h
Poor Design: 20 km/h $< |V_{85i} - V_{85i+1}|$

where V_{85i} = operating speed on section i (km/h).

For achieving sound transitions between successive design elements, the recommended ranges for good, fair, and poor design practices are determined on the basis of the absolute difference in V_{85}. *Good design* practice means that consistency in horizontal alignment exists between successive design elements and that the horizontal alignment does not create inconsistencies in vehicle operating speed. *Fair design* practice means that these road sections may contain at least minor inconsistencies in geometric design between successive design elements. Normally, they would warrant traffic warning devices but not redesigns. *Poor design* practice means that these road sections have strong inconsistencies in horizontal geometric design between successive design elements combined with those breaks in the speed profile that may lead to critical driving maneuvers. A noncurvilinear alignment must be expected. Normally, redesigns are recommended.

Safety Criterion II

Good Design: $|V_{85} - V_d| \leq 10$ km/h
Fair Design: 10 km/h $< |V_{85} - V_d| \leq 20$ km/h
Poor Design: 20 km/h $< |V_{85} - V_d|$

where V_{85} = operating speed (km/h), and V_d = design speed (km/h).

Good design practice means that no adaptations or corrections between V_{85} and design speed are necessary. A curvilinear alignment can be expected. *Fair design* practice means that, for example, in the case of 3R projects, superelevation rates should be related to the V_{85} and not to the design speed to ensure that the assumed side friction will accommodate side friction demand. In cases of resurfacing projects, high skid resistance values should be required. *Poor design* practice means that redesigns are usually recommended. A noncurvilinear alignment must be expected. (Note: 6.2 mph = 10 km/h and 12.4 mph = 20 km/h.)

In addition, they calculated the relationship between the radii of successive horizontal curves to meet the stated Safety Criterion I based on speed relationship developed in earlier research (Figure 7). For example, if a curve with a radius of about 1,600 ft (500 m) is combined with a curve with a radius of 400 ft (120 m) then the design is poor. If combined with a radius of 600 ft (180 m), a fair design results. A good design results when the 1,600-ft (500-m) curve is followed by a curve with more than a 700-ft (200-m) radius. The relationships could be updated with more recent speed prediction findings.

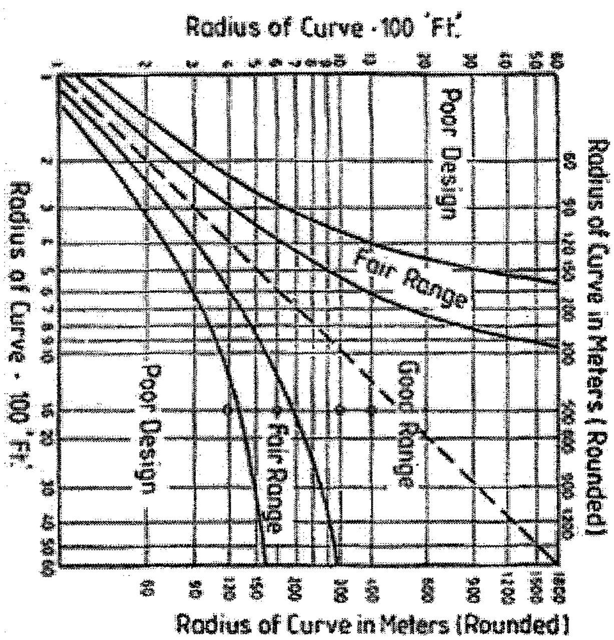


FIGURE 7 Turning of radii of curve sequences for good and fair design as well as for detecting poor design practices on the basis of the U.S. operating speed background (*128*).

Smith and Lamm (*121*) presented numerous indirect visual and safety-related issues so as to assist designers in avoiding horizontal and vertical designs that, in subtle negative ways, may diminish the driver's feeling of comfort, certainty, and safety and that at times may violate the driver's expectations. They recommended that the following be specifically considered for inclusion in the *Green Book*:

- Establishment of appropriate horizontal curvilinear alignment as defined previously for maximum longitudinal grades of up to 5 percent (with exceptions of 6 percent).
- Description of the important three-dimensional (3-D) design elements.
- Increase in the use of perspective plots, which can help to detect numerous visually poor designs. This can be accomplished by using a CADD system now available in most engineering offices.
- A brief discussion on the relationship of the rate of vertical curvature, K, to an equivalent vertical curve radius, R: R (ft) = 100K (feet/percent grade change) or R = 30K, where R is in meters.
- Rules for ratios of horizontal to vertical curve radii (1/5 to 1/10).
- Hilly topography, $R_{crest} > R_{sag}$; flat topography, $R_{sag} > R_{crest}$.
- Rules for the coordination of distortion points.
- Rules for limitations on the length of highway motorists can see.
- Emphasizing the safety benefits of alignment coordination.

Three-dimensional models are used to improve coordination of horizontal and vertical alignments. Several papers discussed the use of computerized 3-D to calculate sight distance (*123–126*). Preview sight distance, which is the sight distance required to see, perceive, and react to a horizontal curve before its beginning, can also be used to assist with coordination of horizontal and vertical alignments (*69,127*). Hassan et al. (*128*) developed an analytical model for headlight sight distance on 3-D combined alignments.

CHAPTER FOUR

CROSS SECTIONS

FREEWAY LANE AND SHOULDER WIDTH

A 1995 NCHRP study (*129*) confirmed that using shoulders and/or narrowing travel lanes could be effective in increasing capacity in congested urban corridors. However, findings indicate that in many instances there may be measurable negative impacts to the overall safety performance of the corridor. These strategies should be reserved for use as techniques for congestion relief, not as a means to widen facilities for extended lengths. Reduction in the travel-lane width to 11 ft (3.4 m) should be the first modification considered. Reduction of the left shoulder should be considered before reducing the right shoulder. Research and observations by enforcement personnel indicate that the right shoulder is the preferred refuge area. Also, emergency response is easier to provide if the right shoulder is maintained. Table 31 summarizes the primary advantages and disadvantages of each approach. The findings of the research have lead to the following recommendations:

- Use of shoulders and narrow lanes to achieve an additional travel lane should *not* normally be considered as an option to a traditional widening project for adding capacity to a freeway corridor.
- For areas of limited length and having turbulent flow conditions, use of shoulder(s) and narrow lanes should be considered as one alternative for achieving smoother flow. Such use should typically be limited to sections of 1 mi (1.6 km) or less.
- Where large truck traffic is a significant proportion of peak period traffic (i.e., 5 to 10 percent), use of shoulders and narrow lanes is not recommended.
- For projects involving possible application of shoulders and narrow lanes, a step-by-step approach (site specific) must be used to ensure an adequate evaluation.

An earlier study (*130*) investigated the effects on accidents of using the inside shoulders as a travel lane. The segments were located in California and most were under 1 mi (1.6 km) in length. There were six cases with no inside shoulders [less than 2 ft (0.6 m) wide] and six with partial inside shoulders [3 to 7 ft (0.9 to 2.1 m) wide] after restriping. Either a nonsignificant change or a significant reduction in overall accidents occurred at all freeway segments except at one site. That site's increase in accidents was determined to be caused by a downstream lane balance issue rather than the shoulder removal. Reduced accidents appear to be related to the lowered levels of congestion. The data suggest that accident reductions occur when the average daily traffic (ADT) per lane change from greater than 20,000 in the before period to less than 18,000 in the after period. The analysis also found that accident severity is not affected; the only significant change is a reduction in noninjury accidents.

MEDIANS

A 1997 NCHRP study (*16,131*) evaluated the operational and safety effects of three alternative median treatments. The study focused on "mid-signal" performance. The approach used in the study was to collect field data to calibrate the evaluation methodology and then use the calibrated methodology to develop treatment selection

TABLE 31
PRIMARY ADVANTAGES AND DISADVANTAGES OF DESIGN ALTERNATIVES (*129*)

Design Alternative	Advantages	Disadvantages
Use of Left Shoulder	• Left shoulder not used as much for emergency stop/or emergency enforcement • Least expensive if width is available • Trucks often restricted from left lane	• Usually requires re-striping • Sight distance problem with some median strips
Use of Right Shoulder	• Often the easiest to implement	• Right shoulder is preferred area for emergency stops and enforcement • Sight distance changes at merge and diverge areas of ramps
Use of Both Shoulders	• Not recommended • Use *only* in extreme cases	• Requires re-striping • Safety concerns (no refuge) • Enforcement is difficult • Incident response longer • Maintenance more difficult and expensive

TABLE 32
CONVERSION FROM AN UNDIVIDED CROSS SECTION TO A RAISED-CURB MEDIAN (*131*)

Average Daily Traffic (vpd)	Active Access Point Density[a] ap/mi (ap/km)	Left-Turn Percent per 1,320 ft (402.6 m) Segment Length[b]					
		0	5	10	15	20	30
17,500	30 (48.3)	U	U	U	U	U	
	60 (96.6)	U	U	U			
	90 (144.9)	U					
22,500	30 (48.3)	U					R
	60 (96.6)						R
	90 (144.9)					R	R
27,500	30 (48.3)			R	R	R	R
	60 (96.6)			R	R	R	R
	90 (144.9)		R	R	R	R	R
32,500	30 (48.3)		R	R	R	R	R
	60 (96.6)		R	R	R	R	R
	90 (144.9)		R	R	R	R	R
37,500	30 (48.3)		R	R	R	R	R
	60 (96.6)	R	R	R	R	R	R
	90 (144.9)	R	R	R	R	R	R
42,500	30 (48.3)	R	R	R	R	R	
	60 (96.6)	R	R	R	R	R	R
	90 (144.9)	R	R	R	R	R	R

[a]Access point (ap) density represents the total number of active access points on both sides of the street segment (i.e., a two-way total) divided by the length of the segment (in miles or kilometers). An active access point is defined as a driveway or street having an entering volume of 10 vph or more.
[b]Total number of left-turns per hour exiting the major street into an access point in one direction of travel per 1,320-ft (402.6-m) length of roadway divided by the total flow rate in that direction (expressed as a percentage).
Legend:

U Stay with existing undivided cross section.
Site-specific examination required.
R Consider adding a raised-curb median.
Volume levels may yield congested conditions.

guidelines. The operations and safety models were used to develop guidelines for selecting a median treatment. The performance measures predicted by these models were used to compute the road-user benefit associated with a change in median treatment. This benefit was then compared to the construction cost associated with the treatment conversion. Arterial conditions that were (and were not) found to be cost-effective were identified in the selection guidelines. The resulting guidelines for four-lane arterial streets in business and office areas are shown in Tables 32–34. Guidelines for six-lane arterials and for residential and industrial areas are available in *NCHRP Report 395* (*16*).

Several other projects also investigated the performance of various median treatments. The following is a list of some of the key findings from those projects concluded during the 1990s:

- An analysis of 3 years of accident data (1991–1993) from 189 street segments located in Phoenix, Arizona, and Omaha, Nebraska, was conducted (*132*), and indicated that accidents are more frequent on street segments with higher traffic demands, driveway densities, or public street densities. It also found that the undivided cross section has a significantly higher accident frequency than the two-way left-turn lane (TWLTL) or raised-curb median treatments when parallel parking is allowed on the undivided street. When there is no parking allowed on either street, the difference between the undivided and the TWLTL treatments is generally small and is negligible for ADT demands of less than 25,000 vpd. The raised-curb median treatment appears to be associated with fewer accidents than the undivided cross section and the TWLTL, especially for ADT demands in excess of 20,000 vpd.
- Venigalla et al. (*133*) used TRAF-NETSIM to simulate performance of TWLTLs and nontraversable medians on four-lane roadways. The measures of operational effectiveness were delay and fuel consumption. The analysis found that at low driveway density and low traffic volume, the difference in total delay between the two designs is not found to be significant. At higher driveway densities, no significant difference in delay to left-turning traffic on the arterial can be expected between TWLTLs and nontraversable medians. However, TWLTL design is found to cause less delay to through traffic and to be more fuel-efficient at all levels of driveway density and traffic volume.
- In 1994, Parsonson et al. (*134*) reported on the safety effectiveness of replacing a TWLTL with a raised median on a high-volume, six-lane arterial in Atlanta. They found a 37 percent reduction in total accident rate and a 48 percent drop in the injury rate for the 4.34 mi (7.0 km) section.

TABLE 33
CONVERSION FROM AN UNDIVIDED CROSS SECTION TO A TWO-WAY LEFT-TURN LANE (*131*)

Average Daily Traffic (vpd)	Active Access Point Density[a] ap/mi (ap/km)	Left-Turn Percent per 1,320 ft (402.6 m) Segment Length[b]					
		0	5	10	15	20	30
17,500	30 (48.3)	U	U	U	U	U	U
	60 (96.6)	U	U	U	U	U	U
	90 (144.9)	U	U	U	U	U	U
22,500	30 (48.3)	U	U	U	U	U	
	60 (96.6)	U	U	U	U	U	
	90 (144.9)	U	U	U	U	U	
27,500	30 (48.3)	U	U				T
	60 (96.6)	U	U			T	T
	90 (144.9)	U	U		T	T	T
32,500	30 (48.3)	U		T	T	T	T
	60 (96.6)	U		T	T	T	T
	90 (144.9)	U		T	T	T	T
37,500	30 (48.3)	U	T	T	T	T	T
	60 (96.6)	U	T	T	T	T	T
	90 (144.9)	U	T	T	T	T	T
42,500	30 (48.3)	U	T	T	T	T	
	60 (96.6)	U	T	T	T	T	T
	90 (144.9)	U	T	T	T	T	T

See Table 32 for notes a and b.
Legend:
U Stay with existing undivided cross section.
Site-specific examination required.
T Consider adding a two-way left-turn lane.
Volume levels may yield congested conditions.

TABLE 34
CONVERSION FROM A TWO-WAY LEFT-TURN LANE TO A RAISED-CURB MEDIAN (*131*)

Average Daily Traffic (vpd)	Active Access Point Density[a] ap/mi (ap/km)	Left-Turn Percent per 1,320 ft (402.6 mi) Segment Length[b]					
		0	5	10	15	20	30
17,500	30 (48.3)						
	60 (96.6)						
	90 (144.9)						
22,500	30 (48.3)						
	60 (96.6)						
	90 (144.9)						
27,500	30 (48.3)						
	60 (96.6)	R	R	R			
	90 (144.9)	R	R	R	R	R	R
32,500	30 (48.3)	R	R	R	R	R	R
	60 (96.6)	R	R	R			
	90 (144.9)	R	R	R	R	R	
37,500	30 (48.3)	R	R	R	R	R	R
	60 (96.6)	R	R	R			T
	90 (144.9)	R	R	R	R		T
42,500	30 (48.3)	R	R	R	R	R	R
	60 (96.6)	R	R	R		T	T
	90 (144.9)	R	R	R			T

See Table 32 for notes a and b.
Legend:
T Stay with existing two-way left-turn lane.
Site-specific examination required.
R Consider adding a raised-curb median.
Volume levels may yield congested conditions.

Bowman and Vecellio (*135,136*) wrote two reports that investigated the relationship between pedestrian safety and alternative median designs. A literature search, a state-of-the-practice survey, and an accident analysis were conducted. The accident analysis included 32,894 vehicular and 1,012 pedestrian accidents occurring in three cities on arterials with raised median, TWLTL, or undivided cross section. Some of the findings from the accident analysis are

- Pedestrian accident rates for central business district (CBD) locations and undivided arterials were significantly higher than those for arterials with raised-curb and TWLT medians. Pedestrian accident rates for arterials with raised-curb medians were lower than those for arterials with TWLT medians and undivided cross sections in CBD locations.
- Arterials with raised-curb medians in suburban areas had the lowest pedestrian accident rate. Arterials with raised-curb medians had a significantly lower pedestrian accident rate than arterials with undivided cross sections. There was no significant difference between the pedestrian accident rates on arterials with raised-curb and those with TWLT medians.
- Study results indicate that, when possible, arterials with undivided cross sections should not be used in CBD areas. In CBD areas, undivided arterials result in the highest accident rates for both pedestrians and vehicles.

The authors note that although the results of the safety analysis on medians and refuge islands are mixed, it appears that both raised and TWLT medians significantly reduce the number and severity of vehicular accidents. The literature review made it apparent that both raised and TWLT medians offer significant vehicular accident reductions and vehicular benefits over those for comparable roadways without medians. Typical reductions in the total number of vehicular accidents for both median types are in the 25 to 35 percent range. The literature did not provide a conclusive indication that medians improved pedestrian safety. This was because of the small number of pedestrian accidents encountered during the studies. The state-of-the-practice survey revealed that there is no universal set of factors that can be used to determine the need to install medians. Whereas states rely on accident history, traffic volumes, numbers and locations of driveways, type of access control, and cost, the larger cities rely on traffic volumes, available right-of-way, and street classification. A greater divergence was found in the smaller cities.

UTILIZATION OF STREET WIDTH

Knapp et al. (*137*) investigated issues associated with the conversion of urban four-lane undivided roadways to a three-lane cross section. They utilized past research and case study experiences in their project. They recommended the following actions:

- The feasibility of replacing an urban four-lane undivided roadway with a three-lane cross section should be considered on a case-by-case basis.
- The present and future characteristics of each of the factors discussed in Knapp et al. (*137*) should be investigated to determine the design period feasibility of converting an urban four-lane undivided roadway to a three-lane cross section.
- A conversion will be most successful if the factors that define the roadway environment remain stable during the design period (e.g., traffic volumes will not increase dramatically) and the current four-lane undivided roadway is already operating as a "de-facto" three-lane roadway.
- More formal, consistent, and widespread before-and-after studies of this type of conversion should be completed and documented.
- The expected operational impacts of converting an urban four-lane undivided roadway to a three-lane cross section should be modeled and documented.
- Formal guidelines for the feasibility, installation, and evaluation of a three-lane cross section versus a four-lane undivided or wider cross section should be published.
- If a three-lane cross section is determined to be feasible it should be considered, along with other alternatives, within a detailed engineering study for comparison purposes.
- Transportation professionals should consider the three-lane cross section as just one more possible improvement alternative for urban four-lane undivided roadways.

In *NCHRP Report 330*, Harwood (*138*) determined the effectiveness of various alternative strategies for reallocating the use of street width on urban arterials without changing the total curb-to-curb width. He found that the safety of urban arterials could be improved by implementing strategies that involve the use of narrower lanes (Table 35). The research addressed urban arterial streets with curb-and-gutter cross sections and speeds of 45 mph (72.5 km/h) or less. Guidelines for the implementation of improvement projects on existing urban arterial streets were developed based on the results obtained in the research study and the experiences of the highway agencies that participated in the study. The guidelines address many of the nonquantitative issues in successful implementation of improvement strategies for urban arterial streets, especially those involving narrower lanes. He notes that the guidelines are intended to supplement, rather than supersede, existing design policies such as those of AASHTO and individual state highway agencies. The following is a list of

TABLE 35
ACCIDENT REDUCTION EFFECTIVENESS OF SELECTED PROJECT TYPES ON URBAN ARTERIAL (*138*)

Project Type	Accident Rate Reduction	
	Expected Value (%)	90% Confidence Interval (%)
Conversion from a four-lane undivided street to a five-lane street with a TWLTL	44	13–75
Conversion from a four-lane divided street with a narrow median to a five-lane street with a TWLTL	53	24–82
Conversion from a six-lane divided street with a narrow median to a seven-lane street with a TWLTL	24	11–38

Note: TWLTL = two-way left-turn lane.

guidelines for projects involving narrower lanes on urban arterials.

- Narrower lane widths [less than 11 ft (3.4 m)] can be used effectively in urban arterial street improvement projects in which the additional space provided can be used to relieve traffic congestion or address specific accident patterns. Narrower lanes may result in increases in some specific accident types, such as same-direction sideswipe collisions, but other design features of a project may offset or more than offset that increase.
- Projects involving narrower lanes nearly always reduce accident rates when the project is made to implement a strategy known to reduce accidents, such as the installation of a center TWLTL or removal of curb parking. Highway agencies should not hesitate to implement such projects on urban arterial streets.
- Projects involving narrower lanes whose purpose is to reduce traffic congestion by providing additional through lanes may result in a net increase in accident rates, particularly at intersections. Such projects should be evaluated carefully on a case-by-case basis, considering the agency's previous experience with that type of project. Both the traffic operational and traffic safety effects of the project should be evaluated and the feasibility of incorporating geometric improvements at intersections (such as left-turn lanes) to reduce intersection accidents should be considered.
- Lane widths as narrow as 10 ft (3.1 m) are widely regarded by urban traffic engineers as being acceptable for use in urban arterial street improvement projects. Except for one specific project type that is not common (conversion from a two-lane undivided to a four-lane undivided street), all projects evaluated in this study that consisted exclusively of lane widths of 10 ft (3.1 m) or more resulted in accident rates that were either reduced or unchanged. Where streets cannot be widened, highway agencies should give strong consideration to the use of 10-ft (3.1-m) lanes, where they are necessary, as part of a geometric improvement to upgrade traffic operations or alleviate specific accident patterns.
- Lane widths less than 10 ft (3.1 m) should be used cautiously and only in situations in which it can be demonstrated that increases in accident rates are unlikely. For example, numerous project evaluations in this study found that 9- and 9.5-ft (2.7- and 2.9-m) through-traffic lanes can be used effectively in projects to install a center TWLTL on existing four-lane undivided streets. Such projects nearly always result in a net reduction in accident rate. On streets that cannot be widened, highway agencies should consider limiting the use of lane widths of less than 10 ft (3.1 m) to (1) project types in which their own experience indicates that they have been used effectively in the past or (2) locations where the agency can establish an evaluation or monitoring program for at least 2 years to identify and correct any safety problems that develop.
- In highly congested corridors, agencies should anticipate that traffic operational improvements on one street, such as the provision of additional through lanes, might attract traffic to that street from parallel streets. This may lead to increased traffic volumes and increased accidents on the improved street, but may still reduce delays and accidents in the corridor as a whole.
- Projects that change the geometries of signalized intersection approaches should be accompanied by adjustments in signal timing (and, in some cases, changes in signal phasing). Traffic volumes on the project (and, possibly, on parallel streets) should be reviewed 1 or 2 months after project implementation to determine if there is a need for further adjustments in a signal timing.
- Truck volumes are an important consideration in the implementation of projects involving narrower lanes. There appears to be general agreement that narrower lanes do not lead to operational problems when truck volumes are less than 5 percent. Sites with truck volumes between 5 and 10 percent should be evaluated carefully on a case-by-case basis. Use of narrower lanes should be discouraged on streets with more than 10 percent truck traffic.
- Higher truck volumes may not cause operational problems on streets with narrower lanes if the trucks travel straight through the site without turning.

- Trucks may be a greater concern on streets with horizontal curves than on tangents.
- Tractor-trailer combination trucks may be more critical than single-unit trucks because of their greater width and their greater offtracking.
- Curb lanes should usually be wider than other lanes by 1 to 2 ft (0.3 to 0.6 m) to provide allowance for a gutter and for greater use of the curb lanes by trucks. Center or left lanes for through traffic and TWLTLs can usually be narrower than the curb lane. One city engineer has pointed out that the left lane for through traffic on an arterial street can be quite narrow if it is adjacent to a center TWLTL, which increases the "effective width" of the through lane. The presence of a TWLTL adjacent to a through lane is obviously less restrictive than the presence of a curb or another through lane.
- Narrow lane projects do not work well if the right lane provides a rough riding surface because of poor pavement conditions or the presence of grates for drainage inlets. Drivers may avoid the right lane if they believe uncomfortable driving will occur over rough drainage inlets. Thus, projects with narrower lanes may be most satisfactory at sites with curb inlets that do not have grates in the roadway.
- The needs of bicyclists should be considered in implementing projects involving narrower lanes. The literature indicates that curb lane widths of at least 15 ft (4.6 m) are desirable to accommodate the shared operation of bicycles and motor vehicles; thus, it may not be possible to fully accommodate bicyclists even on many existing streets with 12-ft (3.6-m) curb lanes. Decisions concerning implementation of projects with narrower lanes should be made by taking into consideration the number of bicyclists using the roadway and the availability of other bicycle facilities in the same corridor.
- When lanes are narrow, operational efficiency at some sites may be reduced because of staggering of traffic in adjacent lanes. The capacity per lane may be reduced because drivers are reluctant to travel side by side. However, drivers in adjacent lanes still travel at shorter headways than they could in a single lane, so the overall through traffic capacity of the street should increase, but not by as much as would be possible if wider lanes could be used.
- Projects that can be implemented by remarking only can be implemented very quickly, often in a single day. However, projects that involve construction, such as median removal, require more time to complete.
- A common problem in remarking projects is that it is difficult to remove the existing pavement markings completely. Current removal methods include grinding, sandblasting, and waterblasting. Because of these problems, some agencies implement almost all remarking projects in conjunction with pavement resurfacing.
- Remarking projects may be confusing to drivers if the new lane lines no longer match the pavement joint lines (or the reflections of the pavement joint lines). This potential problem is another indication that implementation of remarking projects in conjunction with pavement resurfacing is very desirable.
- Access control regulations concerning driveway location and design are important on all urban arterial streets, but especially for streets that are not wide enough to install a median or center TWLTL. Driveway design and location measures that have been found to be effective are summarized in *NCHRP Report 330* (*138*).

Harwood states that the lessons to be drawn from this experience are that traffic operational and safety problems are related, and solving traffic operational problems on an arterial highway can lead to safety benefits as well. Guidelines for performing evaluation of the urban arterial street improvement projects are also provided.

A study on the operation and safety of bicycle lanes versus wide curb lanes used videotape data on 4,600 bicyclists in Santa Barbara, California; Gainesville, Florida; and Austin, Texas. Significant differences in operational behavior and conflicts were found between bike lanes and wide curb lanes, but varied depending on the behavior being analyzed. Wrong-way riding and sidewalk riding were much more prevalent at wide curb lane sites compared with bike lane sites. The overall conclusion from the study is that both bike lanes and wide curb lane facilities can and should be used to improve riding conditions for bicyclists. The identified differences in operations and conflicts appear to be related to the specific destination patterns of bicyclists riding through the intersection areas studied and not to the characteristics of the bicycle facilities. Three documents were produced from the study: a final report (*139*) containing a complete discussion of the research method, data collection procedures, and data analysis; an implementation manual (*140*); and a guidebook (*141*) about innovative bicycle accommodations.

The safety effects of curbs on high-speed suburban multilane highways were evaluated using 10 before-and-after Texas sites and 9 matched-pair Illinois sites (*142*), and include the following recommendations:

- When driveway density is low and traffic volume high, curbs and gutters may not be a wise option, because it may increase accident rates and result in unsafe road conditions. On roadways with high driveway densities, however, curbs and gutters may help the road to operate more safely.

- Installation of curbs and gutters on a high-speed suburban multilane roadway requires special attention to drainage design to prevent undesirable stormwater ponding. Placement of inlets, adequate cross-section sloping, and minimum grade requirements are all considerations in drainage design.
- Nighttime lighting to increase the visibility of the road section should be considered. This lighting would allow the nighttime driver to see the line of the curb.
- Researchers recommended performing a future study that would include a thorough examination of thresholds where driveway density and traffic volume show the safety benefits, or lack of safety benefits, that curbs and gutters have on a high-speed roadway.

HIGH-OCCUPANCY VEHICLE LANES

The most recent work on high-occupancy vehicle (HOV) lanes is contained in *NCHRP Report 413* (*143*) and *NCHRP Report 414* (*144*). *NCHRP Report 413* (*143*) documents gaps and weaknesses in the current practices for developing or expanding HOV systems. It presents an implementation plan for transferring the completed *HOV Systems Manual* into practice. *NCHRP Report 414* (*144*) is a comprehensive and detailed HOV systems manual that incorporates current guidelines and practices. This manual includes the following 13 chapters:

1. Guide to the *HOV Systems Manual*
2. Introduction to HOV Facilities
3. Policy Considerations with HOV Facilities
4. Planning HOV Facilities
5. Operation and Enforcement of HOV Facilities on Freeways and in Separate Rights-of-Way
6. Design of HOV Facilities on Freeways and in Separate Rights-of-Way
7. Operation and Enforcement of Arterial Street HOV Facilities
8. Design of Arterial Street HOV Facilities
9. Transit and Support Services and Facilities
10. Supporting Program and Policies
11. Implementation Consideration with HOV Facilities
12. Public Involvement and Marketing Programs
13. Monitoring and Evaluating HOV Facilities

TRUCKS

Harkey et al. (*145*) conducted a study to determine the differences in performance between 102-in. (259.1-cm) and 96-in. (243.8-cm) wide trucks. Truck data were collected on rural two-lane and multilane roads that included curve and tangent sections and a variety of roadway widths and traffic conditions. The results of the study revealed that the wider trucks had significantly higher rates of edgeline encroachments and tended to drive closer to the centerline than the narrower trucks. Trucks encroach over the edgeline more frequently and to a greater degree where wide paved shoulders exist (i.e., the drivers use the paved shoulders as additional lane width). Some trucks encroach over the edgeline even when little or no paved shoulder exists. The authors recommended that the use of 12-ft (3.7-m) lanes and a minimum of 3-ft (0.9-m) paved shoulders should be considered on rural roadways carrying truck traffic consisting of both 96-in. (243.8-cm) and 102-in. (259.1-cm) wide trucks. Providing paved shoulders of 3 ft (0.9 m) or more will significantly increase construction costs on many roadway sections; therefore, a benefit–cost analysis is needed to determine the economic feasibility of such shoulder construction projects. The authors noted that such an analysis requires information on the accident effects of such improvements related to trucks and other vehicles, and that such effects could not be quantified in their study.

Harwood et al. (*146*) estimated the extent of geometric design improvements and the costs of those improvements to accommodate particular truck configurations on particular roadway networks. The costs were estimated for truck configurations that are larger than the 48-ft (14.6-m) tractor-semitrailer combination used as the baseline vehicle for the study. Substantial costs would be required to accommodate potential future trucks on the existing roadway system. These costs are sensitive to the size of the truck and the extent of the roadway system considered.

LOWER SPEED ROADWAYS

Geometric design dimensions for several traffic calming measures used on low-speed roadways are provided in *Traffic Calming State of the Practice* (*147*). Dimensions are provided for speed humps, speed tables, traffic circles, and roundabouts, along with seven designs from Canada for diagonal diverter, semi-diverter, forced turn island, median barrier, chicane, raised crosswalk, and raised intersection.

Fwa and Liaw (*148*) developed an approach to select the geometric dimensions of a hump based on the design 85th percentile hump-crossing speed and a peak vertical acceleration that governs a drivers' choice of hump-crossing speeds. An example application is presented in which the hump-crossing speeds predicted using the proposed design procedure are found to be in good agreement with hump-crossing traffic speeds measured in Singapore and the United Kingdom. The authors cautioned that additional verification and more evidence are needed before the Singapore-based data for speed-control hump design are applied in other regions.

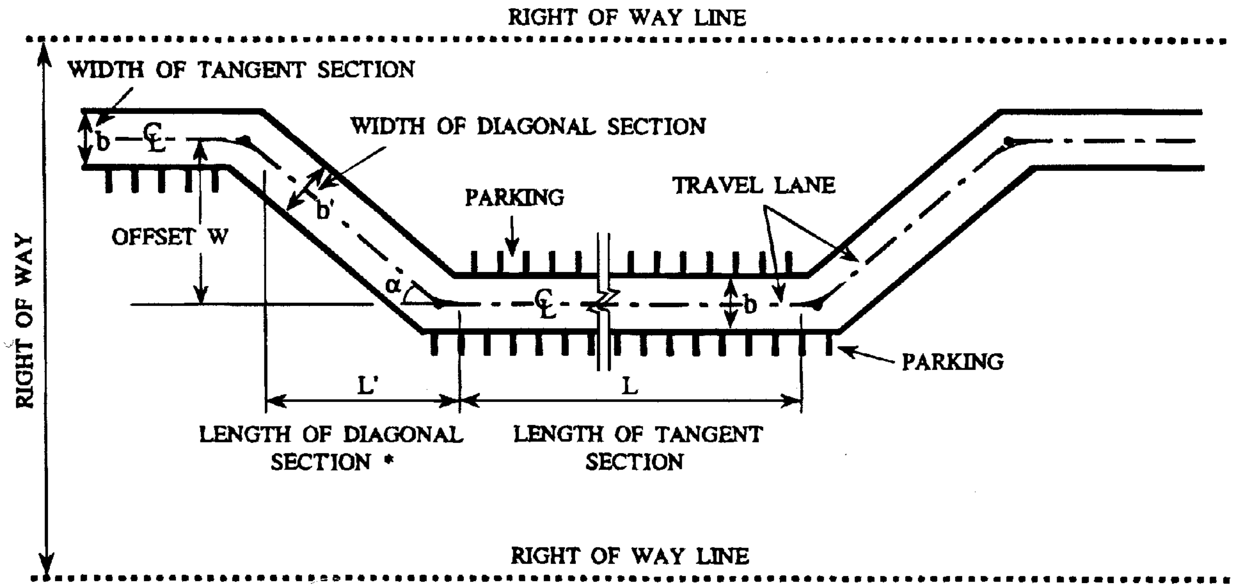


FIGURE 8 General shape and definitions of a shared-street alignment (*149*).

It should be noted that the U.S. Access Board has received numerous complaints that speed humps and other vertical deflection measures have a detrimental effect on the health of some people with disabilities. The "Building a True Community" (*35*) report by the PROWAAC includes discussions on traffic calming and other issues.

Polus and Craus (*149*) discussed the "shared streets" concept, which provides an area that is shared by pedestrians and vehicles. The area is designed to allow only one vehicle to pass through a straight section and two vehicles can meet in the diagonal section. The angle of the diagonal section and the length and width of the straight section are the primary parameters that strongly affect the travel speed along a shared street. The geometry determines the path of vehicles that results in a possible safe speed for a given design. Figure 8 shows the general shape and definitions of a shared street–street alignment.

Gattis and Watts (*150*) investigated the relationships among urban street function (arterial and local), width, and resulting speed. Crash data for the roadways were also considered. The objective of the study was to determine if the wider streets did have more objectionable traits (e.g., higher speeds or crash rates) than did the narrower streets. Six two-lane streets in a small city were considered. The findings suggested that street width may play a small role in vehicle speed, but other factors such as trip function may be more significant determinants of the average and 85th percentile through vehicle speeds. The authors state that the relationships among two-lane urban street function, width, and resulting speed cast doubt on statements that narrower streets automatically result in lower mean or 85th percentile speeds. From the limited number of examples studied, the authors suggest that observed speeds are related more to the drivers' ability to travel a considerable distance before expecting to stop.

SUBURBAN ARTERIALS

Fambro et al. (*151*) developed geometric design guidelines for suburban, high-speed curb, and gutter roadways based on safety, operational, and clear zone studies. Design elements addressed included design speed, alignment, cross section, drainage, driveways, and sight distance. Field data collection sites for the studies were selected from various areas throughout the state of Texas. The safety studies analyzed the safety effects of high-speed curb and gutter roadway sections through accident rates, accident severities, and characteristic frequencies. The operational studies included a study concerning shoulder requirements and TWLTL requirements. The clear zone study was conducted to determine the most appropriate and cost-beneficial clear zone width requirements for suburban high-speed curb and gutter sections. Figure 9 reproduces the recommended English units guidelines (recommendations were also developed in metric units). The resultant guidelines were based on the input from a panel of experts and the results of several safety, operational, and computer simulation studies. They were prepared in a format for possible insertion into the edition of the Texas DOT's *Design Division's Operations and Procedure Manual* (*152*) current in 1995. Some of the material was revised by the Texas DOT prior to inclusion.

SAFETY

In 1992, the FHWA published the third volume of the report, *Safety Effectiveness of Highway Design Features*

4-301 A GENERAL

The term "suburban highway" as used in this publication refers to high-speed (50 to 55 mph) roadways which serve as transitions between low speed (45 mph and below) urban streets and high speed rural highways (i.e., suburban highways connect urban streets to rural highways). These facilities are typically 1 to 3 miles in length and have light to moderate driveway densities (approximately 10 to 30 driveways per mile). Because of their location, suburban highways have both rural and urban characteristics. For example, these sections typically maintain high speeds (a rural characteristic) while utilizing curb and gutter to facilitate drainage (an urban characteristic). Consequently, guidelines for suburban highways typically fall between those for rural highways and urban streets.

4-302 A BASIC DESIGN FEATURES

Figure 4-26 A shows tabulated basic geometric design criteria for suburban highways. The basic design criteria shown in Figure 4-26 A reflect minimum and desired values which are applicable to projects on new location or major improvement projects.

Figure 4-26 A. Refers to Paragraph 4-302 A. Geometric Design Criteria for Suburban Highways

Item	Functional Class	Desired	Minimum
Design Speed	All	60	50
Max. Horizontal Curvature (degrees)	All	see Figure 4-4	see Figure 4-4
Max. Gradient (%)	All	see Figure 4-14	see Figure 4-14
Stopping Sight Distance (ft)	All	—	200
Width of Travel Lane	Arterial	12	11[1]
	Collector	12	10[2]
Curb Parking Lane Width (ft)	All	None	None
Shoulder Width[3] (ft)	All	10	4
Width of Refuge Lanes[4] (ft)	All	11–12	10
Offset to Face of Curb (ft)	All	4[5]	2
Median Width (ft)	All	see Sec. 4-302A(B)1 & 2	see Sec. 4-302A(B)1&2
Border Width (ft)	Arterial	12	8[6]
	Collector	11	8[6]
Right-of-Way Width (ft)	All	Determined by Local Conditions	Determined by Local Conditions
Sidewalk Width (ft)	All	6–8[7]	4
Superelevation	All	Yes	None
Clear Zone Widths (ft)	All	See Sec. 4-302A(G)	See Sec. 4-302A(G)
Vertical Clearance for New Streets (ft)	All	16.5	16.5[8]
Turning Radii	All	See Sec. 4-710(D)	See Sec. 4-710(D)
Structure Widths (ft)	All	Curb face-to-curb fact plus sidewalk(s)	Curb face-to-curb fact plus sidewalk(s)

[1]In highly restricted locations, 10 ft permissible.
[2]In industrial areas 12 ft usual, and 11 ft minimum for restricted right-of-way conditions. In nonindustrial areas, 10 ft minimum.
[3]For ADT > 5,000, shoulders provide significant benefit. For ADT < 3,000, shoulders provide no significant benefit.
[4]Applicable when right- or left-turn lanes are provided.
[5]Applicable for areas with concentrated bicycle traffic.
[6]Depends on clear zone requirements.
[7]Applicable for commercial areas, school routes, or other areas with concentrated pedestrian traffic.
[8]Exceptional cases near as practical to 16.5 ft, but never less than 14.5 ft. Existing structures that provide at least 14 ft may be retained.

A. Access Control

A major concern for suburban highways is the large number of access points introduced due to commercial development. This creates conflicts between exiting/entering traffic and through traffic. In addition, the potential for severe accidents is increased due to the high-speed differentials. Driver expectancy is also violated because through traffic traveling at high speeds does not expect to have to slow down or stop. Research has shown that reducing the number of access points and increasing the amount of access control will reduce the potential for accidents. In addition, accident experience can be reduced by separating conflicting traffic movements with the use of turn bays and/or turn lanes.

Access driveways shall be installed in accordance with the departmental publication, *Regulations for Access Driveways to State Highways*.

FIGURE 9 Geometric design guidelines for suburban, high-speed curb and gutter roadways (*151*).

B. Medians

Medians are desirable for suburban highways with four or more lanes primarily to provide storage space for left-turning vehicles. Medians may be curbed or flush.

1. Curbed Medians
Raised medians with curbing are used on suburban arterials where it is desirable to control left-turn movements. These medians should be delineated with curbs of the mountable type. Curbed medians are applicable on high-volume roadways with high demand for left turns. Curbed medians should be minimally 12 ft (10-ft storage lane plus 2-ft divider at restricted locations), and desirably up to 18 ft (12-ft storage lane plus 6-ft divider) in width.

2. Flush Medians
Flush medians may include pavement markings delineating directional turning bays, or they may be used where appropriately marked as continuous two-way left-turn lanes (TWLTL). The TWLTL design allows use of the flush median area for left turns by traffic from either direction. The TWLTL is applicable on suburban highways with moderate traffic volumes and low-to-moderate demands for left turns. For suburban highways, TWLTL facilities should minimally be 14 ft and desirably 16 ft in width.

The usual value of 16 ft in width should be used on new location projects or on reconstruction projects where widening necessitates the removal of exterior curbs. The "minimum" value of 14 ft in width is appropriate for restrictive right-of-way projects and improvement projects where attaining "usual" median lane width would necessitate removing and replacing exterior curbing to gain only a small amount of roadway width.

To warrant the use of a continuous TWLTL on a suburban highway, the following three criteria should be met:
1. ADT volume of 3,000 or more,
2. Side road plus driveway density of 10 or more entrances per mile, and
3. Length of three-lane section of 1.5 miles or less.

C. Borders and Sidewalks

The border, which accommodates sidewalks, utilities, etc., and separates traffic from privately owned areas, is the area between the roadway and right-of-way line. Every effort should be made to provide wide borders to serve functional needs, reduce traffic nuisances to adjacent development, and for aesthetic reasons. Sidewalks should be a minimum of 4 ft in width, with increased widths applicable near schools, commercial areas, or other areas with high pedestrian volumes. A 2-ft separation should be provided between the backside of curb and the edge of sidewalk. Border widths minimally are 8 ft and desirably 12 ft or more.

D. Bicycle Facilities

Generally, on high-speed roadways minimum shoulder and curb-offset widths are adequate to accommodate expert riders. If high bicycle volumes are anticipated, or volumes with less experienced riders, separate facilities should be considered. Additional guidance is presented in AASHTO's *Guide for the Development of Bicycle Facilities.*

E. Grade Separations and Interchanges

Although grade separations and interchanges are infrequently provided on suburban highways, they may be the only means available for providing sufficient capacity at critical intersections. Normally, a grade separation is part of an interchange (except grade separations with railroads); it is usually of the diamond type with four legs. Locations considered include high-volume intersections and intersections where terrain conditions favor separation of grades.

F. Right-of-Way Width

The width of right-of-way for suburban highways is influenced by traffic volume requirements, land use, cost, extent of ultimate expansion, and availability. Width is the summation of the various cross-sectional elements, including widths of travel lanes, shoulders, median, sidewalks, and borders.

G. Clear Zone

Guidelines for clear zone widths for suburban highways are developed based on the benefit--cost approach. The basic concept behind this approach is that funds should only be invested in projects where the expected benefits would equal or exceed the expected direct costs of the project. Figure 4-27A presents the general clear zone guidelines for suburban highways. The information is intended to provide general guidance to the highway engineer in the selection of appropriate clear zone widths for high-speed curb and gutter sections.

The recommendations contained in this figure are rather straightforward. For each of the four different ADT ranges, the minimum and desirable clear zone widths are provided. For example, for roadways with ADT between 8,000 and 12,000, the recommended minimum and desirable clear zone widths are 10 ft and 20 ft, respectively. Due to the probabilistic nature of the benefit–cost analyses and the assumptions inherent therein, some flexibility in the application of this information is considered acceptable and a certain degree of judgment should be exercised.

FIGURE 9 (*Continued*).

H. Intersections

The number, design, and spacing of intersections influence the capacity, speed, and safety on suburban highways. Capacity analysis of signalized intersections is one of the most important considerations in intersection design. Dimensional layout or geometric design considerations are closely influenced by traffic volumes and operational characteristics and the type of traffic control measures used.

Due to the high operating speeds (50 mph or greater) on suburban highways, curve radii for turning movements should equal that of rural highway intersections (see Section 4-710); however, spare restrictions due to right-of-way limitations in suburban areas may necessitate reduction in the values given for rural highways. Where heavy volumes of trucks or buses are present, increased curb radii of 30 or 50 ft expedite turns to and from through lanes. Where combination tractor-trailer units are anticipated in significant volume, reference should be made to the material in Section 710.

In general, intersection design should be rather simple, free of complicated channelization, to minimize driver confusion. Sight distance is an important consideration even in the design of signalized intersections since, during the low-volume hours, flashing operation may be used.

I. Speed-Change Lanes

Depending on available cross sections and due to high operating speeds on suburban highways, speed-change lanes may be provided as space for deceleration and acceleration from intersecting side streets with significant volumes.

Figure 4-28A shows taper and storage lengths for left-turn lanes on suburban highways. A short curve is desirable on each end of the taper, but may be omitted for construction ease. Where reverse curves are used, the intervening tangent should be one-third to one-half of the total taper length, and the turnoff curve should be about twice the radius of the second curve.

J. Parking

Desirably, parking adjacent to the curb on suburban highways should not be allowed.

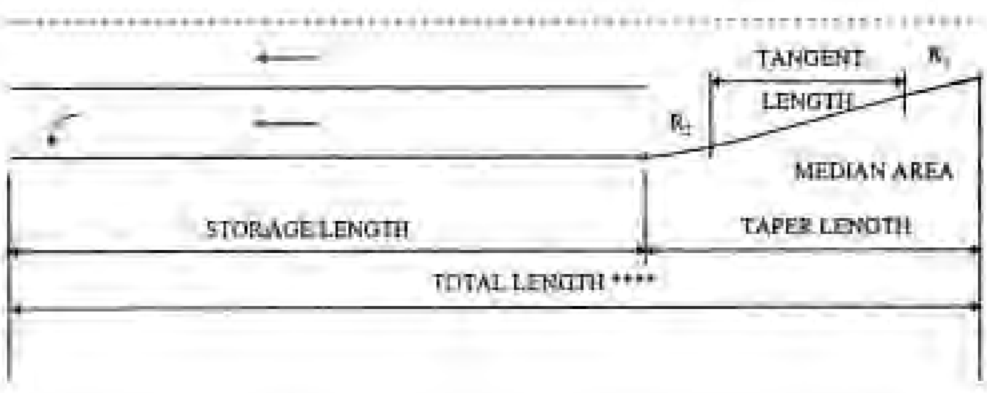
Figure 4-27 A. General Clear Zone Guidelines.

ADT	Recommended Clear Zone Distance,[a] ft	
	Minimum	Desirable
<8,000	10	10
8,000–12,000	10	20
12,000–16,000	10	25

[a] Purchase of 5 ft or less of additional right-of-way strictly for satisfying clear zone provisions is not cost–beneficial and thus not required.

Length of Left-Turn Lanes—Suburban Highway*

Design Speed (mph)	Minimum Taper Length (ft)	Storage Length (ft) **			
		Signalized		Unsignalized	
		Min.	Des.	Min.	Des.
50	180	***	320	100	320



$R_1 = 2R_2$ (Approx.)
Tangent Length = (1/3 to 1/2) (taper length)

Note: Taper length and storage length from table.
* Applicable to speed-change lanes to accommodate left or U-turns at median opening.
** Block spacing may dictate lesser values.
*** Based on design hour volume; storage length = 0.19 to 0.25, multiplied by left-turn peak hour volume.
**** Total length of left-turn lane = storage length + taper length.

Figure 4-28 A. Refers to Paragraph 4-302A(I)

FIGURE 9 (*Continued*).

TABLE 36
ACCIDENT REDUCTION FACTORS DUE TO REDUCING ROADSIDE HAZARD RATING (*154*)

Reduction in Roadside Hazard Rating[a] (No. of Levels)	Reduction in Related Accidents[b] (%)
1	19
2	34
3	47
4	52
5	65

[a]Roadside hazard rating is based on a seven-point hazard scale with steep sideslopes and/or large obstacles close to the roadway corresponding to a hazard rating of seven, and clear, level roadsides representing a hazard rating of one.
[b]Related accidents include head-on, opposite-direction sideswipe, same-direction sideswipe, and run-off-road accidents.

TABLE 37
ACCIDENT REDUCTION FACTORS DUE TO INCREASING ROADSIDE CLEAR RECOVERY DISTANCE (*154*)

Amount of Increased Roadside Recovery Distance[a], ft (m)	Reduction in Related Accidents[b] (%)
5 (1.5)	13
8 (2.4)	21
10 (3.1)	25
12 (3.7)	29
15 (4.6)	35
20 (6.1)	44

[a]Roadside recovery distance is the relatively flat, unobstructed area adjacent to the outside edge of the shoulder within which there is a reasonable opportunty for safe recovery of an out-of-control vehicle.
[b]Related accidents include head-on, opposite-direction sideswipe, same-direction sideswipe, and run-off-road accidents.

TABLE 38
SUMMARY OF EXPECTED PERCENT REDUCTION IN SINGLE-VEHICLE ACCIDENTS DUE TO SIDESLOPE FLATTENING (*154*)

Sideslope Ratio in Before Condition	Sideslope Ratio in After Condition				
	3:1	4:1	5:1	6:1	Flatter than 6:1
2:1	2	10	15	21	27
3:1	0	8	14	19	26
4:1	—	0	6	12	19
5:1	—	—	0	6	14
6:1	—	—	—	0	8

concerning cross sections (*153*). It provided a summary of identified relationships between cross-sectional elements and accident experience. The following cross-sectional elements were included: lanes and shoulders, roadside recovery distance/clear zone, sideslopes, utility poles, other obstacle types, bridges, median design, and multilane design alternatives.

Zegeer et al. (*154*) determined the effects of various roadside features on accident experiences. The information was developed for use by highway agency officials in determining where roadside improvements are justified. The benefits of various roadside improvements can be determined by using the information described. By estimating the costs for roadside improvements such as sideslope flattening, removing trees, and relocating utility poles, the cost-effectiveness may be determined.

Detailed traffic, accident, roadway, and roadside data were collected on 4,951 mi (7971.1 km) of two-lane rural roads in seven states. Statistical analyses and log-linear modeling were used to determine the effects of various roadside and roadway features on single-vehicle and other related accident types. Accident reduction factors were developed based on reducing roadside hazard rating (Table 36) and roadside clear recovery distance (Table 37). Based on the model results for various sideslopes, Table 38 was developed to show likely reductions in single-vehicle accidents due to various sideslope-flattening projects. It indicates that flattening a sideslope of 2:1 on a two-lane rural highway would be expected to reduce single-vehicle accidents by 2 percent if flattened to 3:1, 10 percent if flattened to 4:1, and 27 percent if flattened to 7:1 or flatter.

Another analysis involved the types of roadside obstacles that are most commonly struck for roads with various traffic volume conditions. The frequency of six types of fixed-object accidents for different ADT categories is summarized in Table 39, based on data from six states. For roads with ADTs of 4,000 or less, trees are the single most common type of obstacle struck. This may simply be because trees are generally the most common type of obstacle along low-volume rural roads. For roads with ADTs of over 4,000, utility poles are the single most frequent type

TABLE 39
FIXED-OBJECT ACCIDENTS BY ADT GROUP AND TYPE OF OBSTACLE STRUCK ON URBAN AND RURAL HIGHWAYS (*154*)

	No. of Accidents (Percent of accidents by ADT class)							
ADT Group	Trees	Signs	Utility Poles	Mail Boxes	Bridge Ends	Guard Rail	Other Obstacles	Total
50–400	31 (24.0)	6 (4.7)	2 (1.6)	2 (1.6)	1 (0.8)	5 (3.9)	82 (63.6)	129 (100.0)
401–750	92 (23.7)	20 (5.2)	24 (6.2)	10 (2.6)	5 (1.3)	20 (5.2)	217 (55.9)	388 (100.0)
751–1,000	107 (22.4)	9 (1.9)	26 (5.4)	6 (1.3)	2 (0.4)	33 (6.9)	295 (61.7)	478 (100.0)
1,001–2,000	278 (15.8)	95 (5.4)	118 (6.7)	46 (2.6)	33 (1.9)	192 (12.9)	997 (56.7)	1,759 (100.0)
2,001–4,000	467 (15.8)	200 (6.8)	319 (10.8)	144 (4.9)	29 (1.0)	319 (10.8)	1,475 (49.9)	2,953 (100.0)
4,001–7,500	483 (13.8)	235 (6.7)	611 (17.5)	198 (5.7)	31 (0.9)	323 (9.3)	1,609 (46.1)	3,490 (100.0)
>7,500	275 (10.9)	198 (7.9)	556 (22.1)	154 (5.8)	31 (1.2)	239 (9.5)	1,070 (42.6)	2,514 (100.0)
Total	1,733 (14.8)	763 (6.5)	1,656 (14.1)	551 (4.7)	132 (1.1)	1,131 (9.6)	5,745 (49.1)	11,711 (100.0)

Note: The database includes 1,741 urban and rural sections in six states.

of fixed object struck, which is logical because higher volume roads are generally in the urban and suburban areas where utility poles are frequently placed near the roadway.

Zegeer et al. (*155*) quantified the benefits expected from lane widening, shoulder widening, shoulder surfacing, and general roadside improvements. Detailed accident, traffic, roadway, and roadside data from 4,951 mi (7971.1 km) of two-lane roads in seven states were collected and analyzed. An accident predictive model and detailed statistical procedures were used to determine expected accident reductions related to various geometric improvements. The following are key study results:

- The types of accidents found to be most related to cross-section features (lane width, shoulder width, shoulder type, and roadside characteristics) include single-vehicle (fixed-object, rollover, or run-off-road) and related multivehicle (head-on, opposite-direction sideswipe, or same-direction sideswipe) accidents. The combination of these accident types was termed *related accidents*.
- The traffic and roadway variables found to be associated with a reduced rate of single-vehicle accidents were wider lanes, wider shoulders, greater recovery distance, lower roadside hazard rating, and flatter terrain.
- The developed accident reductions were based on the detailed analyses and accident predictive models developed for two-lane rural roads having ADTs between 50 and 10,000, lane widths of 8 to 12 ft (2.4 to 3.7 m), and shoulder widths of zero to 12 ft (3.7 m) (paved or unpaved).
- The effects of lane width on related accidents were quantified. The first foot (0.3 m) of lane widening [2 ft (0.6 m) of pavement widening] corresponds to a 12 percent reduction in related accidents, 2 ft (0.6 m) of widening [widening lanes from 9 to 11 ft (2.7 to 3.4 m), for example] results in a 23 percent reduction, and 4 ft (1.2 m) of widening results in a 40 percent reduction.
- The effects of shoulder widening on related accidents was determined for paved and unpaved shoulders. For shoulder widths between zero and 12 ft (3.7 m), the percent reduction in related accidents due to adding paved shoulders is 16 percent for 2 ft (0.6 m) of widening (each side of the road), 29 percent for 4 ft (1.2 m) of widening, and 40 percent for 6 ft (1.8 m) of widening. Adding unpaved shoulders would result in 13, 25, and 35 percent reductions in related accidents for 2, 4, and 6 ft (0.6, 1.2, and 1.8 m) of widening, respectively. Thus, paved shoulders are slightly more effective than unpaved shoulders in reducing accidents.
- Table 40 lists the accident reduction factors for related accident types for various combinations of lane and shoulder widening.
- The study provided a set of accident reduction factors to enable computation of estimated accident benefits for a variety of cross-sectional improvements. The authors recommended that consideration be made for such improvements on all roadway sections being considered for 3R-type projects. An informational guide, *Two-Lane Road Cross-Section Design* (*156*), was developed that enables estimation of the safety benefits of various roadway and roadside improvements on specific sections of two-lane roads.

Turner (*157*) outlined the clear roadside concept and provided specific treatments for several of the most prominent

TABLE 40
PERCENT ACCIDENT REDUCTION OF RELATED ACCIDENT TYPES FOR VARIOUS COMBINATIONS OF LANE AND SHOULDER WIDENING (*155*)

Amount of Lane Widening ft (m)	Existing Shoulder Condition (before period)		Percent Related Accidents Reduced							
			Shoulder Condition in After Period							
			2 ft (0.6 m) Shoulder		4 ft (1.2 m) Shoulder		6 ft (1.8 m) Shoulder		8 ft (2.4 m) Shoulder	
	Shoulder Width ft (m)	Surface Type	Paved	Unpaved	Paved	Unpaved	Paved	Unpaved	Paved	Unpaved
3 (0.9)	0 (0)	NA	43	41	52	49	59	56	65	62
	2 (0.6)	Paved	32	—	43	—	52	—	59	—
	2 (0.6)	Unpaved	34	32	44	41	53	49	60	56
	4 (1.2)	Paved	—	—	32	—	43	—	52	—
	4 (1.2)	Unpaved	—	—	36	32	46	41	54	49
	6 (1.8)	Paved	—	—	—	—	32	—	43	—
	6 (1.8)	Unpaved	—	—	—	—	37	32	47	41
	8 (2.4)	Paved	—	—	—	—	—	—	32	—
	8 (2.4)	Unpaved	—	—	—	—	—	—	39	32
2 (0.6)	0 (0)	NA	35	33	45	42	53	50	61	56
	2 (0.6)	Paved	23	—	35	—	45	—	53	—
	2 (0.6)	Unpaved	25	23	37	33	46	42	55	50
	4 (1.2)	Paved	—	—	23	—	35	—	45	—
	4 (1.2)	Unpaved	—	—	27	23	38	33	48	42
	6 (1.8)	Paved	—	—	—	—	23	—	35	—
	6 (1.8)	Unpaved	—	—	—	—	29	23	40	33
	8 (2.4)	Paved	—	—	—	—	—	—	23	—
	8 (2.4)	Unpaved	—	—	—	—	—	—	31	23
1 (0.3)	0 (0)	NA	26	24	37	34	47	43	55	50
	2 (0.6)	Paved	12	—	26	—	37	—	47	—
	2 (0.6)	Unpaved	14	12	28	24	39	34	48	43
	4 (1.2)	Paved	—	—	12	—	26	—	37	—
	4 (1.2)	Unpaved	—	—	17	12	30	24	41	34
	6 (1.8)	Paved	—	—	—	—	12	—	26	—
	6 (1.8)	Unpaved	—	—	—	—	19	12	31	24
	8 (2.4)	Paved	—	—	—	—	—	—	12	—
	8 (2.4)	Unpaved	—	—	—	—	—	—	21	12

Note: NA = not available.

obstacles. He notes that his paper can be used as an introductory guide for those who want to become familiar with the clear zone concept or as a first step toward developing a clear zone policy. It included suggested procedures for the future and actions for the present. "The first order of business should be the preparation and adoption of statutes, ordinances, standards, and operating policies to fit the local jurisdiction and to minimize future violations of the clear zone." His suggested actions for the present are as follows:

- Concentrate first upon known conditions of high hazard, using historical accident data.
- The second step should be to develop a strategy to inventory roadsides throughout the agency's jurisdiction.
- An inventory should then be conducted, using trained personnel to catalog existing fixed objects.
- Appropriate treatment should be identified for all fixed objects and locations identified during the inventory.
- Priorities should be established for correcting difficult situations. Budgets should be prepared and funding identified. It will take many years to treat all objects in the clear zone, and a priority list is essential to ensure that the most worthy locations are addressed first.
- Where necessary, the public should be warned until the location can be treated.

Zegeer et al. (*158*) quantified the accident effects of lane and shoulder widths on rural roads carrying fewer than 2,000 vehicles per day. The primary database used in the research contained accident and roadway characteristics information for more than 4,100 mi (5500 km) of two-lane roadway sections in seven states. Independent databases from three states for roadways totaling more than 54,000 mi (86 000 km) were selected to validate the accident relationships found in the primary database. The major research conclusions of the study were

- Accident rates on paved, low-volume roads are significantly reduced by wider roadway width, improved roadside condition, flatter terrain, and fewer driveways per 1 mi (1.6 km). No differences in accident rates were found on roads with paved shoulders when compared with the rates on roads with unpaved shoulders. Accident rates are most highly correlated with lane and shoulder widths for single-vehicle and opposite-direction accidents.

- The presence of a shoulder is associated with significant accident reductions for roads with lane widths of 10 ft (3.1 m) or greater. For roads with lane widths of 10 ft (3.0 m), shoulders of 5 ft (1.5 m) or greater are needed to reduce accident rates. For roads with lane widths of 11 and 12 ft (3.4 and 3.7 m), shoulder widths of at least 3 ft (0.9 m) result in significant accident reductions when compared with the number of accidents on roads with narrower shoulders.

The results of the accident data analyses were used along with other considerations in the development of recommended changes to the AASHTO guidelines for roadway widths on low-traffic volume roads. Details of those recommended guidelines are contained in *Roadway Widths for Low-Traffic-Volume Roads* (*159*). Suggested changes include the following:

- Lane widths of 9 ft (2.7 m) may be an appropriate standard for a wider range of operating speeds and traffic volumes than is reflected in the current policy.
- Lane width–shoulder width combinations resulting in a total dimension of 30 to 32 ft (9.2 to 9.8 m) are cost-effective for a greater range of traffic volumes than is reflected in current design policy.
- Justification of full-width [12 ft (3.7 m)] lanes and shoulders [10 ft (3.1 m)] as a basic standard is evident only for roads with higher design speeds, roads with traffic volumes of more than 1,500 vpd, and roads with a significant proportion of heavy vehicle traffic.

A study (*160*), using data from the Highway Safety Information System, examined the effects of various cross-section-related design elements on accident frequency and developed an accident prediction model for rural, multilane, non-freeway highways. The study used accident data from Minnesota for the years 1985 to 1990 for multilane, non-freeway, rural roadway sections of a minimum of 0.3 mi (0.48 km) in length. Pedestrian, bicycle, and animal crashes were eliminated from the database. The model development process yielded the following predictive equation

$$Y = 0.0002 \times (DVMT)^{1.073} \times e^{(0.131X1 - 0.151X2 + 0.034X3 + 0.163X4 + 0.052X5 - 0.094X7 - 0.003X8 + 0.429X9)}$$

where

Y = predicted annual accidents,
$DVMT$ = daily vehicle-miles of travel,
$X1$ = average roadside hazard rating,
$X2$ = access control (partial control = 1, no control = 0),
$X3$ = driveway/mile,
$X4$ = intersections with turn lanes/mile,
$X5$ = intersections without turn lanes/mile,
$X6$ = functional class (rural principal arterial = 1, rural minor arterial/collector = 0),
$X7$ = shoulder width (ft),
$X8$ = median width (ft), and
$X9$ = area location type (rural municipal = 1, rural non-municipal = 0).

The model was developed for a variety of applications, such as (1) developing accident predictions for different rural, multilane highway design alternatives; (2) estimating the accident reductions attributed to changes in the cross section of rural multilane highways; and (3) assessing the potential safety impact of alternative cross sections when upgrading a two-lane rural road to a multilane rural highway.

Michie and Bronstad (*161*) conducted an in-depth study of accident data and estimates of the frequency of unreported accidents to determine a more realistic view of guardrail performance. They determined that unreported guardrail impacts represent approximately 90 percent of the total accidents, with the other 10 percent being reported. Assuming no injuries or fatalities in the unreported drive-away accidents, only 6 percent of all guardrail impacts involve any injury or fatality. Furthermore, analysis reveals that terminals, as opposed to segments of typical lengths, are overrepresented in the accident data, comprising up to 40 percent of the guardrail accidents resulting in fatalities or injuries. Also, clinical data indicate that many of the 6 percentile accidents resulting in injuries or fatalities involve (1) guardrail installations that are obsolete, improperly constructed, or inadequately maintained; (2) noncrashworthy ends; or (3) collisions that are outside of the practical design range of modern guardrail systems. It was concluded that properly installed and maintained longitudinal barriers may successfully perform in 97 to 98 percent of all design range length-of-need impacts, with only 2 to 3 percent of the impacts causing occupant injuries or fatalities; a stark contrast to the erroneous 50 to 60 percent based on only reported accidents.

Opiela et al. (*162*) reported on strategies for improving roadside safety as developed by a group of professionals assembled to review the problem, identify possible solutions, and define impediments to resolving the problem. They structured the possible solutions to the roadside safety problem in the form of a strategic plan. The plan is based on five missions, each having a series of goals and objectives. The missions and goals are listed in Table 41.

TABLE 41
MISSIONS AND GOALS OF THE STRATEGIC PLAN (*162*)

Missions	Goals
Increase the awareness of roadside safety and support for it	• A network of partners • Greater public awareness of the importance of roadside safety • Increased emphasis by partners and better communication between them • Sufficient fiscal resources to address critical needs • Programs to disseminate roadside safety information • Integration of roadside safety in SMS • On-going process for updating the plan
Build and maintain the information resources and analysis procedures	• Improved roadway and roadside inventory data systems • Comprehensive roadway safety information resources • Effective tools and methods for safety analyses • On-going programs to monitor roadside safety
Keep vehicles from leaving the roadway	• Improved highway designs and standards • Improved traffic operating environments • Improved vehicle-based systems to keep driver on the road • Improved driver performance and behavior • Sufficient levels of highway and vehicle maintenance
Keep vehicles from overturning or striking objects on the roadside when they do leave the roadway	• Improved roadway design to reduce vehicle overturning • Improved vehicle design to increase stability • Reduced numbers of hazardous objects on the roadside • Improved driver performance in run-off-the-road situations
Minimize injuries and fatalities when overturns occur or objects are struck in the roadside	• Improved roadside safety hardware • Improved vehicle–roadside compatibility and crash worthiness • Proper selection, design, installation, and maintenance of roadside features • Improved emergency team response • Increased seat belt use and effectiveness

Note: SMS = Safety Management System.

Several strategies and actions were identified including the following examples:

- Install shoulder rumble strips to alert drivers.
- Strategically remove or shield trees or utility poles close to the roadway.
- Use public service announcements and citizens initiatives to increase awareness.
- Improve safety management systems.
- Implement proactive highway maintenance programs.
- Improve driver education programs.
- Increase speed enforcement at locations with known roadside safety problems.
- Promote development of innovative technologies to keep vehicles on the road.
- Improve the proficiency of persons responsible for roadside safety.
- Improve vehicle design to increase compatibility with roadside hardware.
- Improve hardware design.

Council and Stewart (*163*) attempted to estimate the benefits of converting a two-lane rural road to a four-lane undivided or divided facility by developing cross-sectional models producing crash rates for typical sections of two- and four-lane roadways in four different states. The output for the two-lane model was compared with the output from the four-lane model at the same ADT level, and the safety effect was measured as the percentage reduction in crashes per kilometer (per 0.6 m), with the two-lane scenario as the base. Predicted crash reductions for conversion from most typical two- to four-lane divided sections ranged from 40 to 60 percent. The reduction due to conversion to a four-lane undivided configuration is much less well defined, ranging from no effect to perhaps 20 percent. Continuing research needs include (1) verification of the undivided four-lane results, (2) additional information on the effects of driveways, (3) estimates for higher levels of two-lane ADT, (4) expansion of the outcome variable to include crash severity, and (5) verification of all results by before-and-after studies of actual conversions.

An HSIS summary report (*164*) on rolled-in continuous shoulder rumble strips (CSRS) installed on freeways used data for 63 CSRS projects completed between 1990 and 1993 in Illinois [284.1 mi (457.4 km) of rural and urban freeways] and 28 CSRS projects completed between 1988 and 1993 in California [encompassing 122.4 mi (197.1 km) of freeway]. The detailed statistical evaluations included before-and-after evaluations with yoked comparisons and before-and-after evaluations with the comparison group. Separate analyses were also conducted for urban and rural freeways. In each analysis, the CSRS sites were associated with a reduction of single-vehicle run-off-the-road accidents ranging from 21.1 to 7.3 percent.The study estimated that one single-vehicle run-off-the-road accident (at an average cost of $62,200) could be prevented every 3 years based on an investment of $217 to install rolled-in CSRS for 0.6 mi (1 km) and recommended that widespread implementation be considered. The authors cautioned that the current design specifications of the different types of CSRS should be reevaluated given recent concerns raised by bicyclists and operators of emergency and maintenance vehicles. They also suggest that a study of CSRS installed on non-freeways (e.g., two-lane rural roads) should be conducted.

CHAPTER FIVE

INTERSECTIONS

INTERSECTION CONFIGURATIONS

Median Width

Walker (*165*) presented an overview of basic intersection design elements. He reviewed intersection angle; coordination of the vertical profiles of the intersecting roads; coordination of horizontal and vertical alignment for intersections on curves; improvement of operation, safety, and capacity through channelization; and drainage requirements for safe operation. The author also provided several illustrations of the various features discussed in the paper.

Harwood et al. (*166*) examined the relationship of the median width and median opening length to intersection operations and safety by means of accident analyses, field observational studies, and traffic operational modeling. Specific recommendations were made for changes to the *Green Book* as part of the study and included:

- At rural unsignalized intersections on divided highways, medians should generally be as wide as practical, and certainly should be wide enough to accommodate turning and crossing maneuvers by a selected design vehicle, as well as any needed left-turn treatments. In most cases, the appropriate design vehicle for rural unsignalized intersections is a large school bus or a large truck. Whenever possible, the median opening length should be limited to the crossroad pavement width plus shoulders, to better define the turning paths and avoid making the paved area in the median too large.
- At suburban unsignalized intersections, medians should generally not be wider than necessary to provide whatever left-turn treatment is selected. Wider medians at suburban unsignalized intersections are associated with increased accident frequency. At specific intersections where substantial turning and crossing volumes or large vehicles (such as school buses or trucks) are present, highway agencies may find it appropriate to select an appropriate median width to safely store a design vehicle of that type in the median.
- At signalized intersections, medians should not generally be wider than necessary to provide whatever left-turn treatment is needed for current or future traffic requirements. Wider medians at signalized intersections are associated with both increased accident frequency and increased delay.
- Highway agencies should consider limiting median widths at rural unsignalized intersections that are likely to require signalization or undergo suburban development in the foreseeable future. Wider medians should operate well at a rural unsignalized intersection, but may operate poorly if the intersection becomes signalized and/or undergoes development.
- Particular care should be taken in the design and operation of wide-median intersections to ensure that the intersection is properly signed to discourage improper left turns into the near roadway of the divided highway and that, whenever possible, a driver on the crossroad approach to a divided highway intersection can see the far roadway of the divided highway. Both signing and visibility of the far roadway help to discourage wrong-way movements that can lead to accidents. Such intersections should be lighted at night whenever possible. In this research, wrong-way movements were not found to be a major problem; however, the potential for accidents involving wrong-way movements always exists at divided highway intersections.

Roundabouts

In the 1990s, several reports were published on roundabouts. The most recent work and the most comprehensive is an informational guide (*167*) produced by the FHWA. It reflects the best practices from around the world interpreted for use in the United States. This guide includes chapters on policy considerations, planning, operational analysis, safety, geometric design, traffic design and landscaping, and system considerations. Many studies have found that one of the benefits of roundabout installation is the improvement in overall safety performance. Although the frequency of reported crashes is not always lower at roundabouts, reduced injury rates are reported. Safety is better at small and medium capacity roundabouts than at large or multilane roundabouts. Although overall crash frequencies have been reduced, the crash reductions are most pronounced for motor vehicles, less pronounced for pedestrians, and equivocal for bicyclists, depending on the study and bicycle design treatments. Reasons for the increased safety level at roundabouts are listed here.

- Roundabouts have fewer conflict points in comparison with conventional intersections. The potential for hazardous conflicts, such as right-angle and left-turn head-on crashes is eliminated with roundabout use. Single-lane-approach roundabouts produce greater safety benefits than multilane approaches because of fewer potential conflicts between road users, and because pedestrian crossing distances are short.
- Low absolute speeds associated with roundabouts allow drivers more time to react to potential conflicts,

also helping to improve the safety performance of roundabouts.

- Since most road users travel at similar speeds through roundabouts, that is, have low relative speeds, crash severity can be reduced compared with some traditionally controlled intersections.
- Pedestrians need only cross one direction of traffic at a time at each approach as they traverse roundabouts, as compared with unsignalized intersections. The conflict locations between vehicles and pedestrians are generally not affected by the presence of a roundabout, although conflicting vehicles come from a more defined path at roundabouts (and thus pedestrians have fewer places to check for conflicting vehicles.) In addition, the speeds of motorists entering and exiting a roundabout are reduced with good design. As with other crossings requiring acceptance of gaps, roundabouts still present visually impaired pedestrians with unique challenges.

On a planning level, it can be assumed that roundabouts will provide higher capacity and lower delays than all-way stop control, but less than two-way stop control, if the minor movements are not experiencing operational problems. A single-lane roundabout may be assumed to operate within its capacity at any intersection that does not exceed the peak-hour volume warranted for signals. A roundabout that operates within its capacity will generally produce lower delays than a signalized intersection operating with the same traffic volumes and right-of-way limitations. The maximum daily service volume of a single-lane roundabout varies between 20,000 and 26,000 vpd, depending on the left-turn percentages and the distribution of traffic between the major and minor roads. A double-lane roundabout may service 40,000 to 50,000 vpd (*167*).

Other recent documents on roundabouts include syntheses by NCHRP (*168*) and FHWA (*169*) and guidelines developed by the Maryland DOT (*170*) and the Florida DOT (*171*). The NCHRP Synthesis summarizes design guidelines available in the United States, Australia, England, France, Switzerland, and Germany. It should be noted that the U.S. Access Board has received complaints that roundabouts are generally inaccessible to visually impaired pedestrians. Additional discussions regarding roundabouts and people with disabilities are contained in "Building a True Community" (*35*) available online at www.access-board.gov.

Alternative Designs

Polus and Cohen (*172*) investigated the operational impacts resulting from converting a major conventional cross intersection into two smaller signalized intersections. The splitting is a potential design approach for intersections with heavy traffic volumes on major arterials. Polus and Cohen present several advantages, but note that because some disadvantages exist, the use of a split intersection should be viewed generally as an interim solution until such time when an interchange is warranted. They discuss the impacts of a split intersection on capacity and delay and conclude that it is possible to increase capacity and reduce the average delay for vehicles. Both of these positive effects are obtained mainly because of the increase in effective green time, resulting from a reduction in the number of phases, the increase in the number of lanes for storage of vehicles waiting to make a left-turn movement, and a reduction in the area of the smaller junctions compared with the area of the original intersection. The minimum distance between the sub-intersections should allow for the accumulation of the left-turning vehicles in each cycle, and can be determined from Figure 10. Additionally, it is possible to plan for a longer distance to allow for the simultaneous start of the green time at the two individual intersections to allow for the elimination of the queue at the first signal just before the first vehicle from the second signal arrives at the back of that queue. The total delay at the intersection needs to be checked to determine the optimal design.

Bared and Kaisar (*173*) used computer simulation and economic analysis to determine the benefits of split intersections. They found that split intersections are well-suited to alleviate the traffic congestions of single intersections in isolated suburban areas where the total approaching volume is greater than 4,000 vph. It is also possible that split intersections can be used along an arterial with signal progression when the arterial signal synchronization is compatible with the optimal signal timing of the split intersection. Moreover, their application is feasible in urban areas when streets are converted to one-way traffic with adequate available offset. Bared and Kaisar found that a split intersection provides noticeable economic benefits by postponing the construction of the bridge for a grade-separated diamond interchange until traffic growth requires it. For a smooth and economical operation, the length of the split intersection offset and the number and length of left-turn lanes in conjunction with well-coordinated signals are crucial.

Hummer (*174,175*) provided information on unconventional left-turn alternatives for urban and suburban arterials. The alternatives are focused on treating left turns to and from arterials, reducing delay to through vehicles, and reducing or separating the number of conflict points. Hummer notes that by their nature as unconventional solutions and by rerouting certain movements, these alternatives all have the potential to cause more driver confusion than a conventional arterial. However, this can be offset by using the alternatives on a section of the arterial and developing appropriate legible and understandable traffic control devices. Detailed studies on the operation and safety benefits

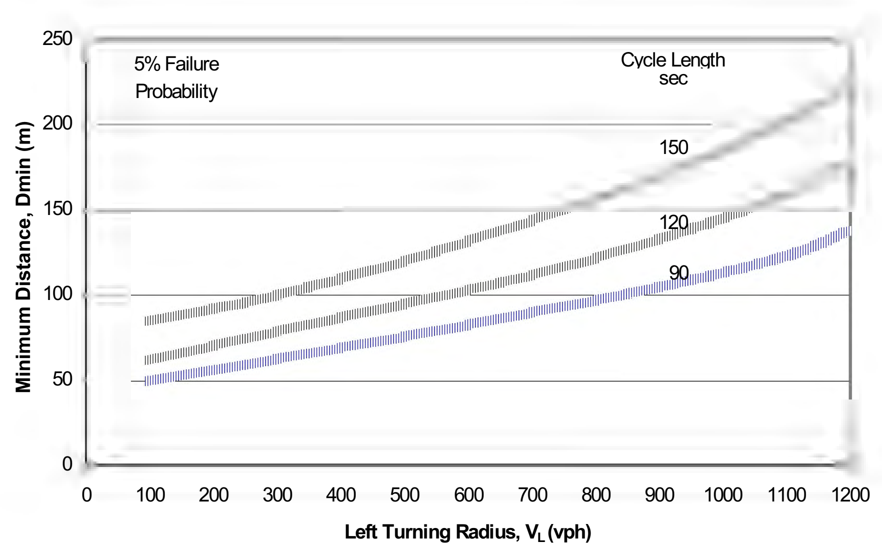


FIGURE 10 Minimum distance between intersections (*172*).

TABLE 42
SUMMARY OF ALTERNATIVE DESIGNS (*175*)

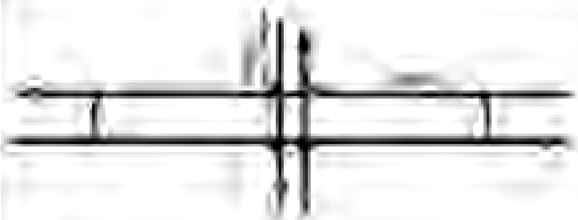
Alternative	Applicable Traffic Volume: Left Turns from Arterial	Applicable Traffic Volume: Left Turns from Minor Street	Applicable Traffic Volume: Minor Street Through	Extra Right-of-Way Needed
Median U-Turn	Low-medium	Low-medium	Any	30 ft (9.2 m) wide along arterial
Bowtie	Low-medium	Low-medium	Low-medium	Two circles up to 300 ft (91.5 m) in diameter on minor street
Superstreet	Any	Low-medium	Low-medium	30 ft (9.2 m) wide along arterial
Paired Intersections	Any	Any	Low	Two 80-ft (24.4-m)-wide parallel collectors
Jughandle	Low-medium	Low-medium	Any	Two 400-ft (122.0-m) by 300-ft (91.5-m) triangles at intersection
Continuous Flow Intersection	Any	Any	Any	Two 400-ft (122.0-m) by 300-ft (91.5-m) rectangles at intersection
Continuous Green T	Any	Low-medium	None	No extra

of the alternatives are not available; however, Hummer noted that the unconventional alternatives, where the number of unprotected conflicting movements has been reduced, are theoretically safer than conventional arterials. Also, agencies have simulation tools available that can determine the benefits of different design alternatives, including unconventional alternatives, for their own arterials. Table 42 summarizes the characteristics of locations that may be suited for an unconventional intersection. Figures 11–17 summarize the information Hummer provided on the seven alternatives.

Reid (*176*) also proposed another unconventional intersection design alternative—the "quadrant roadway intersection" (QRI) design. The QRI design removes left-turn movements from main arterial/cross-street intersections through the use of an additional roadway in one intersection quadrant. Figure 18 shows a typical QRI design and Figure 19 shows the left-turn patterns. By routing all left-turn movements from the arterial and cross street to the quadrant roadway, the main arterial and cross-street intersection can operate with a simple two-phase signal. The spacing of the QRIs from the main intersection is a tradeoff between left-turn travel distance and time versus available storage for the westbound left-turn movement. In the analysis of the QRI, a 491.8-ft (150-m) spacing was selected for both QRIs from the main intersection. Other considerations for QRI include:

- Potential uses of the land within the quadrant roadway such as a service station or convenience store served by right-in/right-out driveways,
- Additional advance signing needs,
- Design modifications for missed left-turn opportunities (consider additional median U-turns beyond the main intersection),
- Preservation of signal operation at each intersection (a fourth intersection leg cannot be developed at either end of the quadrant roadway because these signals must function as T-intersections), and

MEDIAN U-TURN



Description. The median U-turn, shown above, requires left turns to and from the arterial to use directional median crossovers. At a signalized intersection, left turns from the arterial proceed beyond the intersection, make a U-turn at the crossover, and make a right turn back at the main intersection. Left turns to the arterial first make a right turn at the main intersection and then make a U-turn at the crossover. Left turns are prohibited at the main intersection, so the signal there has two phases. The directional crossovers may be signalized, depending on the volumes and other unusual considerations. A signal at a directional crossover near a main intersection should be coordinated with the signal at the main intersection, allowing arterial drivers to stop no more than once. The distance from the main intersection to the nearest crossover is a trade-off between preventing spillback and causing extra driving. Many agencies have found a distance of 600 ft or so works best. Median widths depend on the design vehicle and the radius in which it can make a U-turn. For a large semi-trailer combination design vehicle, the American Association of State Highway and Transportation Officials recommends a median width of 60 ft on a four-lane arterial. A narrower median is possible with a smaller design vehicle, on six- or eight-lane arterials or by providing a paved turning basin beyond the edge line.

Variations. Several variations of the basic median U-turn alternative described above are possible. Placing the directional crossovers on the cross street instead of the arterial minimizes the right-of-way needed along the arterial. Placing directional crossovers on both the arterial and the cross street increases the left-turn capacity. A Stop-controlled directional crossover immediately prior to a main intersection is an interesting variation, which can minimize delay to U-turning traffic. Finally, median U-turns would perform better in states allowing left turns on a red signal from a one-way facility to a one-way facility.

History. The Michigan Department of Transportation (MDOT) is the most prominent user of median U-turns in the United States, with over 1,000 miles in service. Median U-turns have functioned in Michigan for over 30 years; MDOT and other agencies in Michigan continue to design them.

Advantages. The advantages of the median U-turn over a conventional multiphase signalized intersection include:

- Reduced delay for through arterial traffic;
- Easier progression for through arterial traffic;
- Fewer tops for through traffic, particularly on approaches without signalized directional crossovers;
- Fewer threats to crossing pedestrians; and
- Fewer and more separated conflict points.

Disadvantages. The disadvantages of the alternative relative to conventional intersections include:

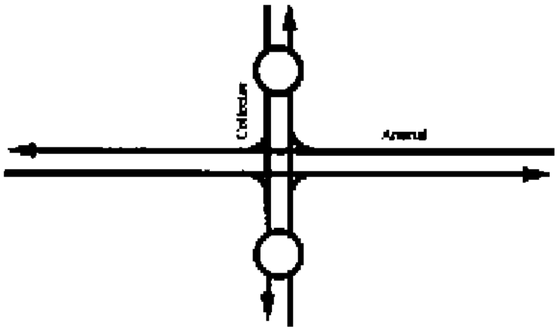
- Driver confusion;
- Driver disregard of the left-turn prohibition at the main intersection;
- Increased delay for left-turning traffic;
- Increased stops for left-turning traffic;
- Larger rights-of-way along the arterial;
- Higher operation costs for extra signals; and
- Longer cross-street minimum green times or two-cycle pedestrian crossing.

In addition, wider means are generally considered to harm roadside businesses. To the extent that conventional arterials can safely and efficiently have narrower medians or two-way left-turn lanes, roadside businesses may benefit.

When to Consider. Agencies should consider the median U-turn alternative where generally high arterial through volumes conflict with moderate or low left-turn volumes and any cross-street through volumes. If the left-turn volume is too high, the extra delay and travel distance for those drivers, and the spillback potential, will outweigh the savings for through traffic. Arterials with narrow medians and no prospects for obtaining extra rights-of-way for widening are poor candidates for the median U-turn, with the exception of cases where agencies can build the wide median and crossovers on the cross street.

FIGURE 11 Alternative designs—Median U-turn (*174*).

BOWTIE



Description. The bowtie alternative, inspired by “raindrop” interchange designs common in Great Britain, is a variation of the median U-turn alternative with the median and the directional crossovers on the cross street. To overcome the disadvantage of requiring a wide right-of-way on the cross street, the bowtie uses roundabouts on the cross street to accommodate left turns instead of directional crossovers across a wide median as shown above. Left turns are prohibited at the main intersection, which therefore requires only a two-phase signal. Vehicles yield upon entry to the roundabout, but if the roundabout has only two entrances as shown above the entry from the main intersection does not have to yield. The roundabout diameter, including the center island and circulating roadway, varies from 90 to 300 ft depending on the speed of traffic on the approaches, the volume of traffic served, the number of approaches, and the design vehicle. The distance from the roundabout to the main intersection could vary from 200 to 600 ft, trading off spillback against extra travel distance for left-turning vehicles. The arterial may have a narrow median. U-turns are difficult, having to travel through both roundabouts and through the main intersection three times, so midblock left turns should be accommodated directly along the arterial.

Variations. A three-legged version of the bowtie is possible but would require much extra right-of-way. It would likely be inferior to a three-legged median U-turn or jughandle except in cases where an agency was later phasing in a fourth leg of the intersection.

History. A few agencies have installed roundabouts on cross streets in an evolutionary manner, but no agency to the author’s knowledge has consciously designed a complete bowtie alternative. Raindrop interchanges, similar to diamond interchanges but with roundabouts instead of signalized or Stop-controlled ramp terminals, have been in use successfully in Great Britain for years. A few raindrop interchanges have been designed and built in the United States recently, most notably in Vail, Colorado.

Advantages. The advantages of the bowtie over conventional multiphase signalized intersections include:

- Reduced delay for through arterial traffic;
- Reduced stops for through arterial traffic;
- Easier progression for through arterial traffic;
- Fewer threats to crossing pedestrians; and
- Reduced and separated conflict points.

Disadvantages. The disadvantages of the alternative relative to conventional intersections include:

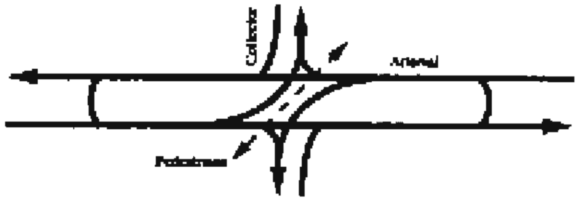
- Driver confusion;
- Driver disregard for the left-turn prohibition at the main intersection;
- Increased delay for left-turning traffic and possibly cross-street through traffic;
- Increased travel distances for left-turning traffic;
- Additional right-of-way for the roundabouts; and
- Difficult arterial U-turns.

When to Consider. Agencies should consider the bowtie alternative where there exist generally high arterial through volumes and moderate to low cross-street through volumes, and moderate or low left-turn volumes. If the left-turn volume is too high, the extra delay and travel distance for those drivers, and the spillback potential, will outweigh the savings for arterial through traffic. Likewise, if the cross-street through volume is too high, delays caused by the roundabout will outweigh the savings for the arterial through traffic. Arterials with narrow or nonexistent medians and no prospects of obtaining extra right-of-way for widening are good candidates for the bowtie. Developers may be convinced with certain incentives to build roundabouts into site planes. The distances between signals should be long so that the extra right-of-way costs for the roundabouts do not overwhelm the savings elsewhere.

Incidentally, roundabouts rarely make sense directly on multilane arterials. Roundabout capacity cannot easily be expanded by widening beyond two lanes, so roundabouts rarely work at intersections between multilane arterials. However, roundabouts are generally inappropriate for intersections between arterials and collectors or local streets because of the extra delay to larger numbers of arterial vehicles.

FIGURE 12 Alternative designs—Bowtie (*174*).

SUPERSTREET



Description. A superstreet is another extension of the median U-turn concept that provides the best conditions for through arterial movements short of interchanges. The superstreet alternative, shown above, requires cross-street through movements and left turns to and from the arterial to use the directional crossovers. Four-approach intersections become two independent three-approach intersections. This independence allows each direction of the arterial to have its own signal timing pattern, including different cycle lengths if desired, so that engineers can achieve "Perfect" progression in both directions at any time with any intersection spacing. Pedestrians can make a relatively safe but slow two-stage crossing of the arterial as shown above. Other design details of the superstreet are identical to median U-turns.

Variations. One variation, at an intersection with a low-volume cross street, is to dispense with the directional crossovers for left turns from the arterial at the intersection. Another variation is to reverse the direction of the crossovers at the intersection to allow left turns to the arterial in cases where those are the heavier volume movements. However, these crossovers create difficult merges for the left on the arterial.

History. Richard Kramer, the long-time traffic engineer in Huntsville, Alaska, USA, conceived of the superstreet alternative and published a paper on it in 1987. To the author's knowledge, nobody has implemented the full superstreet, but some agencies have severed cross-street through movements and built directional crossovers on arterials in a piecemeal fashion.

Advantages. The advantages of the superstreet over a conventional multiphase signalized intersection include:

- Reduced delay for through arterial traffic and for one pair of left turns (usually left turns from the arterial);
- Reduced stops for through arterial traffic;
- "Perfect" two-way progression at all times with any signal spacing for through arterial traffic;
- Fewer threats to crossing pedestrians; and
- Reduced and separated conflict points.

Disadvantages. The disadvantages of the alternative relative to conventional intersections include:

- Driver and pedestrian confusion;
- Increased delay for cross-street through traffic and for one pair of left turns (usually left turns to the arterial);
- Increased travel distances for cross-street through traffic and for one pair of left turns;
- Increased stops for cross-street through traffic and for one pair of left turns;
- A slow two-stage crossing of the arterial for pedestrians; and
- Additional right-of-way along the arterial.

When to Consider. Consider a superstreet where high-arterial through volumes conflict with moderate to low cross-street through volumes. This will be the case for many suburban arterials where roadside development generates most of the conflicting traffic. One should also consider a superstreet where close to 50/50 arterial through-traffic splits exists for most of the day, but uneven street spacings remove any chance of establishing two-way progression. As for median U-turns, arterials with narrow medians and no prospects for obtaining extra rights-of-way for widening are poor candidates for the superstreet.

FIGURE 13 Alternative designs—Superstreet (*174*).

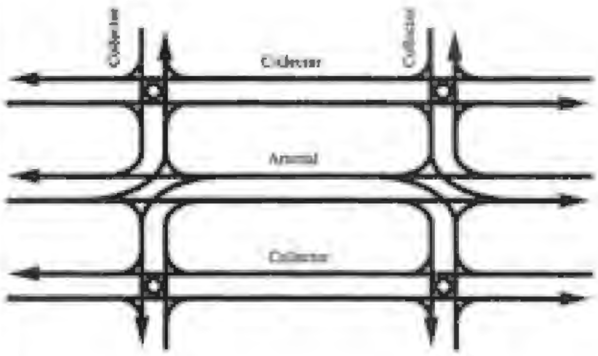
- Restriction of driveways between intersections to preserve left-turn storage for the main intersection approaches.

A CORSIM experiment was conducted that showed improved stopped-delay and system travel time for QRIs as compared to typical arterial intersections. The author noted that while driver expectations may be violated at QRIs, designs similar to QRI have been successfully implemented in the field. Based on his analysis, the identified advantages and disadvantages are listed here.

Advantages:

- Two-phase signal operation at the main intersection creates more progression opportunities by allowing a larger progression bandwidth;
- Reduces total intersection system delay;
- Reduces queuing, especially for the worst approach movements, by greater than 120 percent in level of service conditions;
- Fewer vehicle conflict points at the main intersection and a probable reduction in left-turn or head-on collisions; and

PAIRED INTERSECTION



Descriptions. The paired intersection alternative uses directional crossovers (see above). The alternative employs directional crossovers for left turns from the arterial at one intersection of the pair and directional crossovers for left turns to the arterial at the second member of the pair. Complete circulation throughout the corridor requires that continuous two-way collector roads are parallel to the arterial, are set back at least several hundred feet from the arterial to avoid spill back, and provide developable parcels fronting the arterial. The intersections between the cross streets and the parallel collector roads may be Stop-controlled or signal-controlled depending on the traffic volumes and other usual factors. If developments along the arterial have access from the parallel collector roads, then the arterial median does not have to be wide enough to accommodate U-turns by all vehicles. Like in a superstreet, pedestrians in the paired intersection alternative can make a relatively safe but slow two-stage crossing of the arterial.

Variations. Directional crossovers accommodating left turns to the arterial can operate with a signal controlling both directions of the arterial, but this could make two-way progression suboptimal with poor signal spacing. A variation that preserves perfect two-way progression as in the superstreet is to have the crossover end in a merge onto the arterial, which requires several hundred feet for an acceleration lane and a median that is at least 30 ft or so wide.

History. Agencies have been prohibiting turns from or onto arterials while relying on parallel streets for circulation for years, especially in downtown areas. Designers also have been channeling left turns into a development through one driveway and left turns out of the development through another driveway for years. However, Edison Johnson, a traffic engineer with the city of Raleigh, North Carolina, USA, was the first to conceive of the complete paired intersection alternative (with directional crossovers and parallel collector streets) in the late 1980s when asked to work on a developing arterial where complete conversion to a freeway was not politically acceptable. The design, which appeared in a consultant's report in 1992, is slowly being phased in by the city as the area develops.

Advantages. The advantages of the paired intersection alternative over an arterial with conventional multiphase signalized intersections include:

- Reduced delay for through arterial traffic and for some left turns;
- Reduced stops for through arterial traffic;
- Easier progression for through arterial traffic, and with the left merge variation, "perfect" two-way progression at all times with any signal spacing;
- Fewer threats to crossing pedestrians; and
- Reduced and separate conflict points on the arterial.

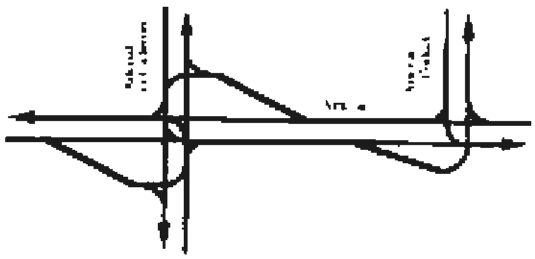
Disadvantages. The disadvantages of the alternative relative to conventional intersections include:

- Driver and pedestrian confusion;
- Increased delay for cross-street through traffic and for some left-turning traffic;
- Increased travel distances for cross-street through traffic and for some left-turning traffic;
- A slow two-stage crossing of the arterial for pedestrians;
- Additional right-of-way for the parallel collector roads; and
- Additional construction, maintenance, and operation costs for the parallel collector roads.

When to Consider. The paired intersection alternative is worth considering for arterials with high through-traffic volumes and low cross-street through volumes. In addition, the means to build and operate the parallel collector roads must be available. In developed corridors, good parallel streets must exist and the environment on them must allow increased traffic. In such circumstances, a one-way pair may be a superior alternative anyway. In developing corridors, agencies may be able to convince developers to pay for a portion of the cost of the collectors, and the agencies should ensure that parcels access the collectors.

FIGURE 14 Alternative designs—Paired Intersections (*174*).

JUGHANDLE



Description. The jughandle alternative uses ramps diverging from the right side of the arterial to accommodate all turns from the arterial. In the four-approach jughandle intersection shown above, the ramps are before the intersection. Left turns from the arterial use the ramp, then turn left on the cross street at the ramp terminal. Ramp terminals are typically Stop-controlled for left turns and Yield-controlled for channelized right turns. In modern jughandles, ramp terminals are several hundred feet from the main intersection to ensure that queues from the signal on the cross street do not block the terminal. Since no U-turns or left turns are allowed directly from the arterial, the median may be narrow. The signal at the main intersection may need a third phase, for left turns from the cross street, if the volume is heavy.

If agencies use jughandles as the only way drivers can make left turns and U-turns along a section of the arterial, all turns will be made from the right lane. This could decrease driver confusion, decrease lane changes, and increase travel speeds in the left lane.

Variations. If left turns from the ramp terminal are difficult, agencies can use loop ramps beyond the main intersection to accommodate left turns from the arterial. The travel distances for the left-turning vehicles are longer with a loop ramp, but loop ramps allow an easier right turn onto the cross street at the ramp terminal. Agencies also can employ loop ramps beyond the intersection for left turns from the cross street to avoid the third-signal phase. Jughandles for three-approach intersections and jughandles exclusively for U-turning traffic use ramps, which curve back to meet the arterial as shown above.

History. The New Jersey Department of Transportation has used jughandles for years on hundreds of miles of heavy-volume arterials and continues to build new jughandle intersections.

Advantages. The advantages of the jughandle alternative over conventional multiphase signalized intersections include:

- Reduced delay for through arterial traffic;
- Reduced stops for through arterial traffic;
- Easier progression for through arterial traffic;
- Narrower right-of-way needed along the arterial; and
- Reduced and separated conflict points.

Disadvantages. The disadvantages of the alternative relative to conventional intersections include:

- Driver confusion;
- Driver disregard for left-turn prohibitions at the main intersection;
- Increased delay for left turns from the arterial, especially if queues of cross-street vehicles block the ramp terminal;
- Increased travel distances for left turns from the arterial;
- Increased stops for left turns from the arterial;
- Pedestrians must cross ramps and the main intersection;
- Additional right-of-way for ramps;
- Additional construction and maintenance costs for ramps; and
- Lack of access to arterial for parcels next to ramps.

When to Consider. Designers should consider jughandles on arterials with high through volumes, moderate-to-low left-turn volumes and narrow rights-of-way. The distances between signals should be long so that the extra right-of-way and other costs for the ramps do not overwhelm the savings elsewhere.

FIGURE 15 Alternative designs—Jughandle (*175*).

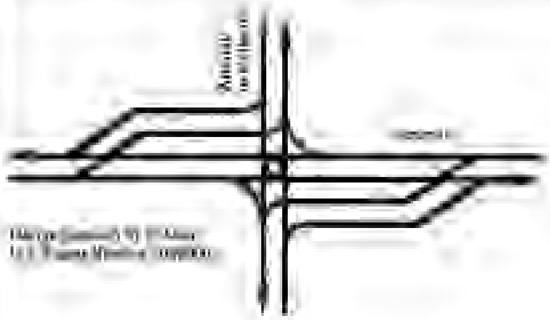
- Narrower intersection widths (by eliminating dual-turn lanes) reduce vehicle clearance and pedestrian crossing times.

Disadvantages:

- Increased left-turn travel distance and the potential for increased left-turn travel times and stops;
- Greater possibility of driver confusion, error at critical (intersection) locations, and missed left-turn opportunities;
- Nonconformity of left-turn patterns for each approach of the same intersection;
- Additional advance signing requirements; and
- Additional right-of-way required for the quadrant roadway.

Reid and Hummer (*177*) quantified the reductions in travel time for the median U-turn (MUT) and superstreet median (SSM) designs over a system of signals as compared to the traditional two-way left-turn lane design using CORSIM. The key function of the MUT design (Figure

CONTINUOUS FLOW INTERSECTION



Description. The continuous flow intersection features a ramp to the left of the arterial upstream of the main intersection to handle traffic turning left from the arterial, as shown above. Usually, high volumes will justify a signal at the crossover where the ramp begins. Engineers can easily coordinate this two-phase signal with the signal at the main intersection. A single signal controls the main intersection and the left-turn ramp/minor-street intersection. The major breakthrough with this design is that arterial through traffic and traffic from this left-turn ramp can move during the same signal phase without conflicting. This allows, in effect, protected left turns with a two-phase signal. The cross-street stop bar must be set back beyond the left-turn ramp, which probably means more lost time and longer clearance intervals for the cross-street signal phase(s). Right turns are removed from conflicts near the intersection with ramps. U-turns on the arterial are possible at the left-turn crossover if the median is wide enough. Without provisions for U-turns the arterial median may be narrow. The left-turn ramp usually crosses the opposing traffic 300 or so feet from the cross street to balance the various higher costs of a longer ramp against the chance of spillback from the main intersection blocking the signal at the crossover.

Francisco Mier of El Cajon, California, USA, holds the U.S. patent, #5049000, for the continuous flow intersection. Agencies wishing to implement the design must contact Mier to obtain the rights.

Variations. If left turns to the arterial are heavy at the continuous flow intersection as shown above, a third-signal phase may be needed at the main intersection. To avoid the third phase, designers can use left-turn ramps in three or all four quadrants of the intersection.

History. Mier obtained his patent in 1987. With co-authors, he has published articles evaluating the concept in general and has written several reports evaluating the concept in particular locations. The first continuous flow intersection in the United States, with ramps in a single quadrant at a T intersection, was opened in 1994 on Long Island, New York, USA, at an entrance to Dowling College. Several others have opened recently in Mexico. Early reports on the operation of these intersections are favorable.

Advantages. The advantages of the continuous flow intersection over a conventional multiphase signalized intersection include:

- Reduced delay for through arterial traffic;
- Reduced stops for through arterial traffic;
- Easier progression for through arterial traffic;
- Narrower right-of-way needed along the arterial; and
- Reduced and separated conflict points.

With ramps in three or four quadrants these advantages may extend to the cross street as well.

Disadvantages. The disadvantages of the alternative relative to conventional intersections include:

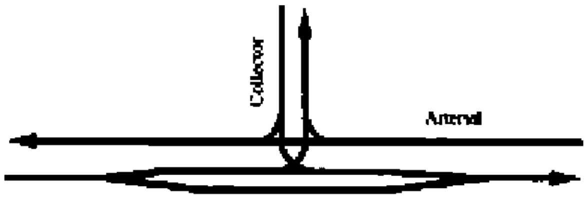
- Driver and pedestrian confusion;
- Increased stops for left turns from the arterial;
- Restricted U-turn possibilities;
- Pedestrians must cross ramps and the main intersection (and pedestrians must cross the four-quadrant design in a slow two-stage maneuver);
- Additional right-of-way for ramps;
- Additional construction, maintenance, and operation costs for ramps and extra signals;
- Lack of access to the arterial for parcels next to ramps; and
- The costs of obtaining the rights to use the design.

If left turns from the arterial experience more delay than at comparable conventional intersections, the extra delay is likely to be small in magnitude.

When to Consider. Agencies should consider the continuous flow intersection on arterials with high through volumes and little demand for U-turns. The designer must have some right-of-way available along the arterial near the intersection and must be able to restrict access to the arterial for parcels near the intersection. Like the bowtie and jughandle alternatives, the extra right-of-way and other costs will be hard to justify if installations are too close together.

FIGURE 16 Alternative designs—Continuous Flow Intersection (*175*).

CONTINUOUS GREEN T



Description. While the other unconventional alternatives discussed worked for both three-approach and four-approach intersections, the continuous green T alternative only works for three-approach intersections (see above). The two through lanes on the top of the T are controlled differently. The median lane is subject to the standard two-phase or (more likely) three-phase signal which also controls opposing through traffic, left turns from the arterial, and turns from the cross street. However, the shoulder lane receives a steady green signal. Pavement marking directs traffic turning left from the cross street into the median lane. Pedestrians must seek signal protection between the two through lanes that can be narrow and should not present a hazardous fixed object, but agencies should make the separation visible and tactile with raised reflectors or rumble strips. The separation should extend several hundred feet upstream and downstream from the intersection to minimize hazardous last-minute weaves. Agencies can use more than one continuous through lane, but dual left-turn lanes from the cross street would mean dual signal-controlled through lanes on the top of the T and would put great pressure on the remaining continuous through lane(s). The arterial should have a raised median of some type, at least for the length of the through-lane separation, to stop vehicles from turning left or from driveways and thereby crossing the through-lane separation.

Variations. The main variation to the continuous green T as shown above is to have all through lanes on the top of the T get a steady green signal while left turns from the cross street are channelized into a merging lane in the median. The merging lane must be lengthy to minimize conflicts. This variation requires a slightly wider median than the minimal 16-ft median (i.e., one exclusive turn lane wide) required for the continuous green T intersection as shown above. The island channelizing the left turns should be very positive; some agencies use curbs with pavement markings and reflectors. A merge from the left is a difficult driving maneuver, so while this variation rewards higher volumes of arterial through traffic it will break down with higher left-turn volumes.

History. Several districts of the Florida Department of Transportation have used the continuous green T alternative shown with no major apparent problems. A large number of agencies use the variation described above.

Advantages. The advantages of the continuous green T intersection over a conventional multiphase signalized T intersection include:

- Reduced delay for through arterial traffic in one direction; and
- Reduced stops for through arterial traffic in one direction.

It is very unlikely that the through movement at the top of the T is a critical movement that controls signal timing. If that movement should happen to be critical, however, removing it from the domain of the signal would lead to reduced delay for all other movements at the intersection.

Disadvantages. The disadvantages of the alternative relative to conventional intersections include:

- Driver and pedestrian confusion;
- Driver disregard of the separation between the through lanes;
- No signal protection for pedestrians to cross the arterial;
- Increased lane changing conflicts before and after the separation of the through lanes; and
- Restricted access to parcels adjacent to the continuous green through lane(s).

Driveways along the continuous green through lane(s) pose two potential problems. First, through drivers in the continuous green lanes may not expect to slow for anything in those lanes, even a right-turning vehicle. Second, drivers turning left onto the arterial from the minor street may try to merge into the continuous green through lane or pass through the lane separation to get to a driveway.

When to Consider. Of the unconventional alternatives discussed in these features, the continuous green T has the most restrictive niche. Engineers should consider it at signalized three-approach intersections with moderate to low left-turn volumes from the minor-street and high-arterial through volumes, where there are no crossing pedestrians and few drivers choose one of the two continuous green lanes.

FIGURE 17 Alternative designs—Continuous Green T (*175*).

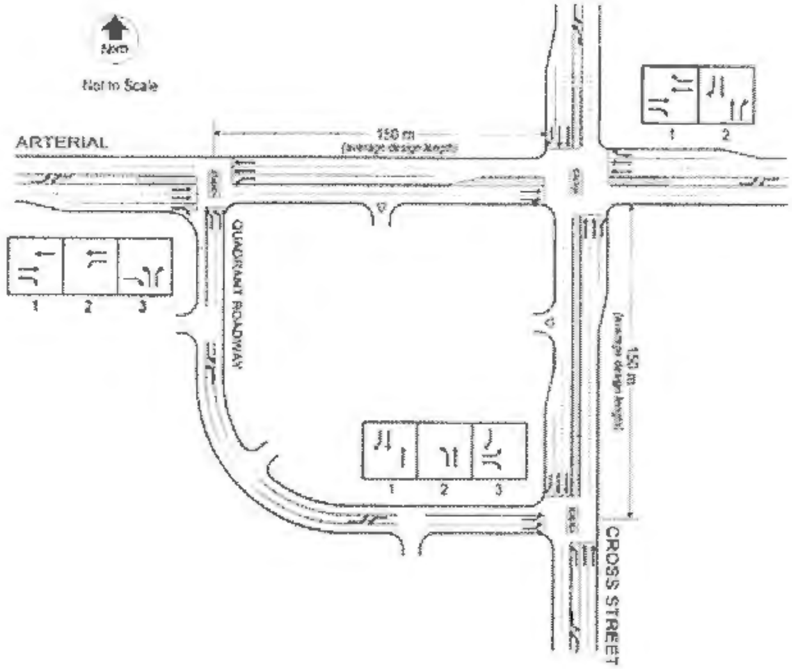
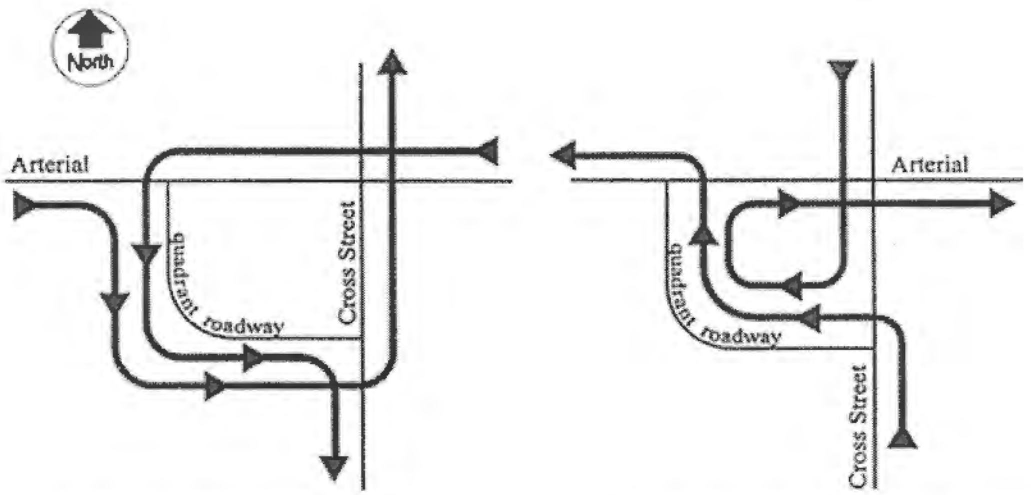


FIGURE 18 QRI design (*176*).



A) Left-Turn Pattern from the Arterial

B) Left-Turn Pattern from the Cross Street

FIGURE 19 QRI left-turn pattern (*176*).

20) is removal of all left-turn movements at signalized intersections, creating two-phase signal operations and increased progression opportunities. The SSM design (Figure 21) allows perfect progression of through traffic in both directions because signals on both sides of the arterial can operate independently. Results showed that the MUT and SSM designs improved system travel time and average speed compared with the TWLTL design. Average speeds for the system increased nearly 25 percent for the MUT alternative and nearly 15 percent for the SSM compared with the TWLTL design. The average number of stops increased for both the MUT and SSM designs. These alternatives should have increased stops because of the increased turning movements required for left-turning vehicles and through vehicles in the case of the SSM design.

Grade-Separated Intersections

Leisch (*178*) presents characteristics of three "grade-separated intersections." He defines grade-separated intersections as referring to the various means of significantly increasing the capacity or resolving physical constraints by grade-separating the through movements on two intersecting roadways and interconnecting the two with ramps or roadways that form one or more intersections. He notes

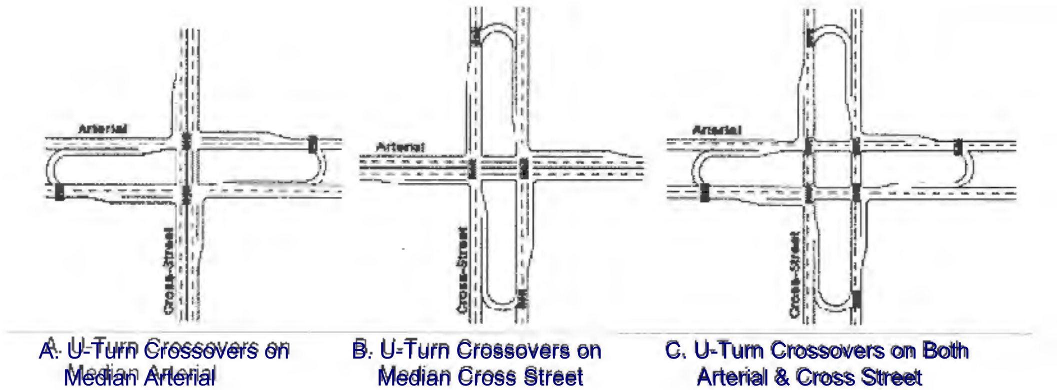


FIGURE 20 Typical median U-turn designs (*177*).

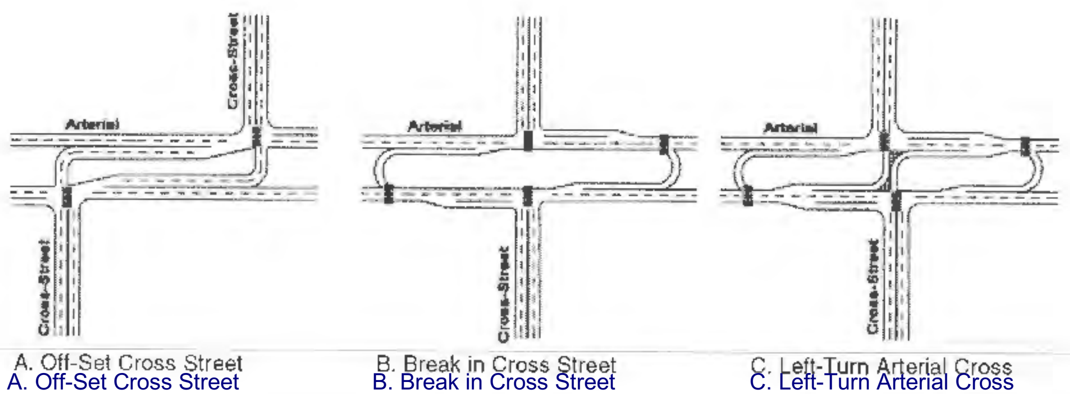


FIGURE 21 Superstreet arterial designs (*177*).

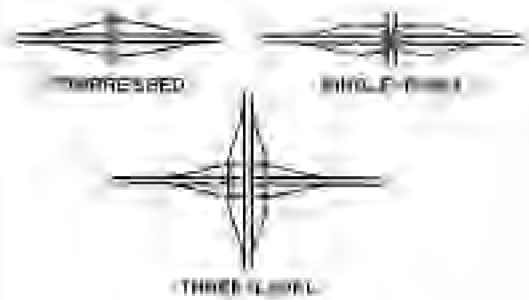
that his paper is intended to supplement the *Green Book* and provide guidance to planners, designers, or traffic engineers in selecting the appropriate forms for a given condition. There are three controls that may dictate the need for a grade separation between two intersecting highway facilities: traffic volumes/capacity, safety, and alignment and profile (terrain). The different forms of grade-separated intersections were categorized as compact diamond, partial cloverleaf (Parclo), and rotary. Figures 22–24 provide characteristics for each of these forms.

Bonilla (*179*) examined the benefits of flyovers, which are defined as grade-separated structures that allow arterial through traffic to go over a crossing arterial or collector without slowing down or stopping for an at-grade signal. He states that capacity per lane is generally that of arterial through lanes, about 1,750 vph. His economic evaluation showed that congested intersections with an approach volume averaged over 20 years of 50,000 vpd or more would justify a simple arterial flyover. The minimum right-of-way for urban arterial flyovers is listed in Table 43; Figure 25 shows minimum cross sections. Safety considerations for flyovers require a smooth transition from at-grade arterial lanes to the flyover. The physical split between exiting intersection-bound traffic and through traffic must be logical, simple, and anticipated.

Figure 26 shows criteria for redirecting traffic lanes. Merging traffic from the at-grade intersection with traffic from the flyover may require somewhat longer tapers, similar to those used for arterial lane drops. Bonilla proposed the following warrants for flyovers:

- The intersection is a bottleneck and conventional traffic engineering measures cannot resolve the capacity problem.
- A minimum of four through lanes already exists and maximum use of the intersection right-of-way has been made. The sum of critical lane volumes approaches or exceeds 1,200 vph.
- It is time-consuming, expensive, or contrary to public objectives to obtain additional right-of-way. A minimum right-of-way of 100 ft (30.5 m) is available.
- Impact to adjacent properties and minor streets limited to right turn only is not severe.
- The accident rate is significantly larger than for nearby intersections on the same arterial.

DIAMOND FORMS



Diamond forms are generally of three types: the single-point diamond, the compressed diamond, and the three-level diamond (see above).

The single-point diamond has the following characteristics:

- It takes little right-of-way.
- It has moderate capacity.
- It is a single intersection with three-phase signal control. Four-phase signal control is required if ramp through traffic movements are provided.
- It is the second most costly to construct of all diamond forms.
- Access is eliminated for a minimum of 1,500 to 2,000 ft along the priority facility.

It is possible to have access on ramps if they are judiciously located.

- U-turn loops to interconnect ramps and reduce intersection traffic can be provided.
- Large left-turn radii can facilitate truck movements.

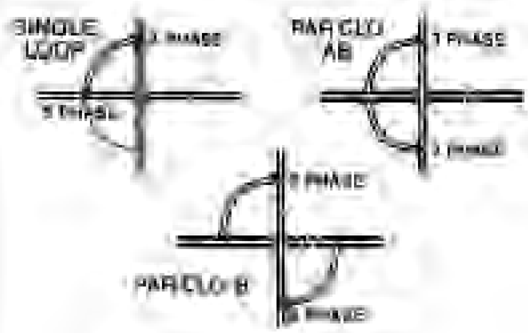
The compressed diamond with ramp terminal intersections 200 to 400 ft apart has some characteristics that are similar to and others that are much different from the single-point diamond.

- It takes the most right-of-way.
- It has high capacity.
- It has four intersections, each with two-phase signal control.
- It is most costly to construct; nearly double the compressed diamond.
- Access is eliminated for a minimum of 1,500 to 2,000 ft along both facilities.
- It is possible to have access on ramps if they are judiciously located.
- There is no need for U-turns loops.

Application and implementation of one of these diamond forms is obviously related to the site-specific requirements associated with traffic/capacity, right-of-way, other physical constraints, and access needs.

FIGURE 22 Diamond forms (*178*).

PARTIAL CLOVERLEAF FORM



Partial cloverleaf forms are also of three general types, as shown above. They include a single-loop and two two-loop varieties. Their characteristics are somewhat different from the diamond forms. Partial cloverleaf forms often have application in locations where physical requirements, right-of-way, constraints, access needs, and highway/street network configurations govern.

The single-loop or "cutoff" roadway form can have many applications in situations where turning traffic movements are not high and when roadway network and access requirements are compatible. This form is often applied where terrain controls and the two-way cutoff roadway is sufficient to accommodate the turning movements between the intersecting roadways.

- It takes little right-of-way.
- It has low to moderate capacity.
- It has two intersections—one on each roadway with three-phase signal control or stop control.
- It is generally not as costly as the diamond forms.
- Access is easy to coordinate; access off loop is possible.
- Consideration of highway/street network is required and could be used.
- It is the lowest in cost of the partial cloverleaf forms.

The Parclo B, two-loop/opposite quadrant form, is somewhat higher in capacity and responds to different traffic pattern and highway network requirements.

- It takes more right-of-way than the single-loop form.
- It has moderate to high capacity.
- It has free flow on the priority street, with right in and right out at connecting roads. Crossroad has two three-phase signal-controlled or Stop-controlled intersections.
- It has construction costs similar to those of the two-loop/opposite quadrant form.
- Access is easy to coordinate; access from both loops is possible.
- Consideration of highway/street network is required and could be used.

FIGURE 23 Partial Cloverleaf forms (*178*).

ROTARY INTERCHANGE FORMS

Rotary interchanges have limited application. There are two types of rotary interchanges, as shown above.

The first configuration (a) may be fitting in suburban areas where a major arterial serves a residential or partly commercial area with multiple streets forming five or more intersection legs and where traffic volumes are of the order that can be accommodated on a series of short weaving sections. The characteristics of a rotary interchange are as follows:

- The right-of-way required is about the same as or slightly more than the three-level diamond.
- It has moderate capacity.
- It has multiple intersections that could operate with yield control and with weaving between them.
- Its construction cost is similar to that of the compressed diamond.

An application of the rotary interchanges is shown in Figure b, in which two arterial highways have all through movements separated, using five structures. Each left-turning movement weaves with the other left-turning movements in negotiating three of the four weaving sections. A rotary with a radius of 400 to 500 ft produces weaving sections about 300 to 400 ft in length. The latter occupy an area approximately equal to a cloverleaf with 150-ft radius loops. In terms of serviceability, each weaving section is limited to a weaving volume of 1,200 to 1,500 vph. Construction cost approaches that of a three-level diamond.

FIGURE 24 Rotary Interchange forms (*178*).

TABLE 43
MINIMUM RIGHT-OF-WAY FOR URBAN ARTERIAL FLYOVERS (*179*)

	Right-of-Way by No. of Lanes, ft (m)		
	Two Lanes	Four Lanes	Six Lanes
Marginal	76 (23.2)	98 (29.9)	
Low type	100 (30.5)	120 (36.6)	140 (42.7)
High type	120 (36.6)	144 (43.9)	168 (51.2)

Another concept for an uncontrolled access urban arterial interchange is the Echelon Interchange (*180*). The Echelon Interchange elevates one-half of a divided highway as it approaches the point of intersection, resulting in two grade-separated intersections. The two grade-separated intersections operate in the same manner as two one-way pair intersections (Figure 27). Miller and Vargas (*180*) concluded that the Echelon Interchange will sometimes, but not always, out perform traditional grade separation designs in signalized networks. It offers two important possible advantages: (1) it will not overpower an adjacent signalized intersection to the extent free-flow movements might; and (2) it offers the planner/designer significant flexibility and more discretionary options relative to its layout and its attendant land-use severance impacts.

INTERSECTION SIGHT DISTANCE

Several papers have examined issues associated with intersection sight distance (ISD) (*181–186*). For example, Easa (*182*) looked at a reliability-based method for ISD design. He noted that the main advantage of the reliability method is that it provides the reliability associated with ISD design values and that it is conceivable that different highway classes could be designed based on different levels of reliability such as having higher-class facilities have larger reliability values than those assumed for lower class facitilies. The most recent work is documented by Harwood et al. in *NCHRP Report 383* (*181*). Field studies to observe driver behavior were conducted at 25 intersections located in 4 states. This work has been accepted by the AASHTO Task Force on Geometric Design and will be incorporated into the next edition of the *Green Book*.

Stop-Control Intersections

The recommendation by Harwood et al. (*181*) for stop-controlled intersections is to base sight distance on gap acceptance values. The leg of the sight triangle 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 44. The sight distance values can be computed with the following equation:

$$ISD = 0.278 \times V \times G$$

where

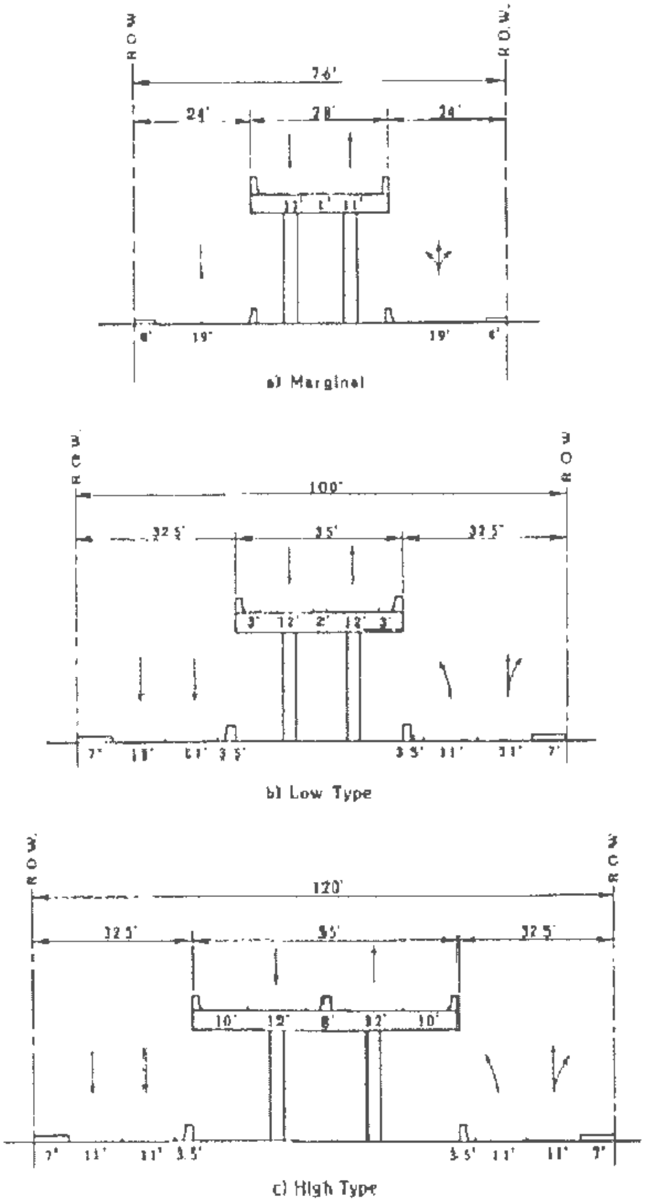


FIGURE 25 Minimum cross section and right-of-way for a two-lane flyover: (a) marginal, (b) low type, and (c) high type (*179*).

ISD = intersection sight distance (m),
V = major road design speed (km/h), and
G = specified critical gap as listed in Table 44 (sec).

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

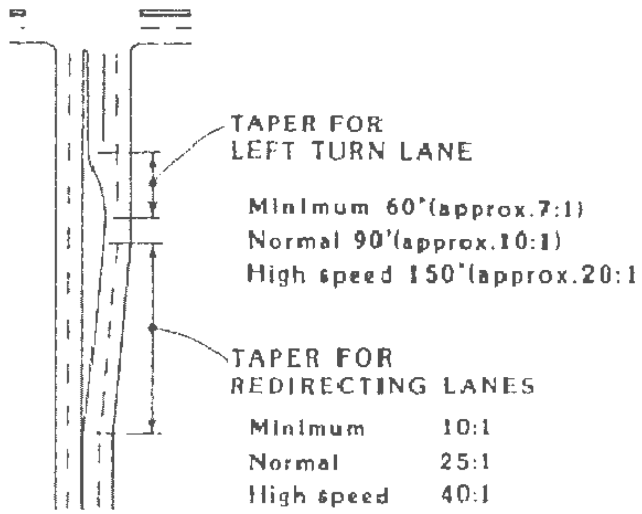


FIGURE 26 Taper design for urban streets (*179*).

in making the left turn. Any median that is at least 12 ft (3.6 m) in width should be considered in determining the number of lanes to be crossed. For example, a 24 ft (7.2 m) 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 3.1 ft (1 m) clearance at each end, then no multilane highway adjustment is necessary and the travel times as shown in Table 44 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.

The leg of the departure sight triangle along the minor road for both left and right turns should generally have a length of 14.4 ft (4.4 m) 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 (*183*).

Uncontrolled Intersections

Harwood et al. (*181*) noted that at uncontrolled intersections a driver should have a view of the intersection from a distance sufficient to stop, if necessary, before reaching the intersection. This is normally assured by the provision of SSD along each of the intersection 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. Table 45 lists the recommended sight distances for uncontrolled intersections. The values were calculated to provide sufficient sight distances for either or both of the approaching vehicles to stop before reaching the intersection with the assumption that drivers slow to 50 percent of the midblock running speed before reaching the intersection. Field observations supported the decision to use 50 percent of the running speed as part of the model. The recommended sight distances presented in Table 45 reflect the anticipated changes to the SSD model as recommended by Fambro et al. (*74*). Harwood et al. (*181*) also noted that no

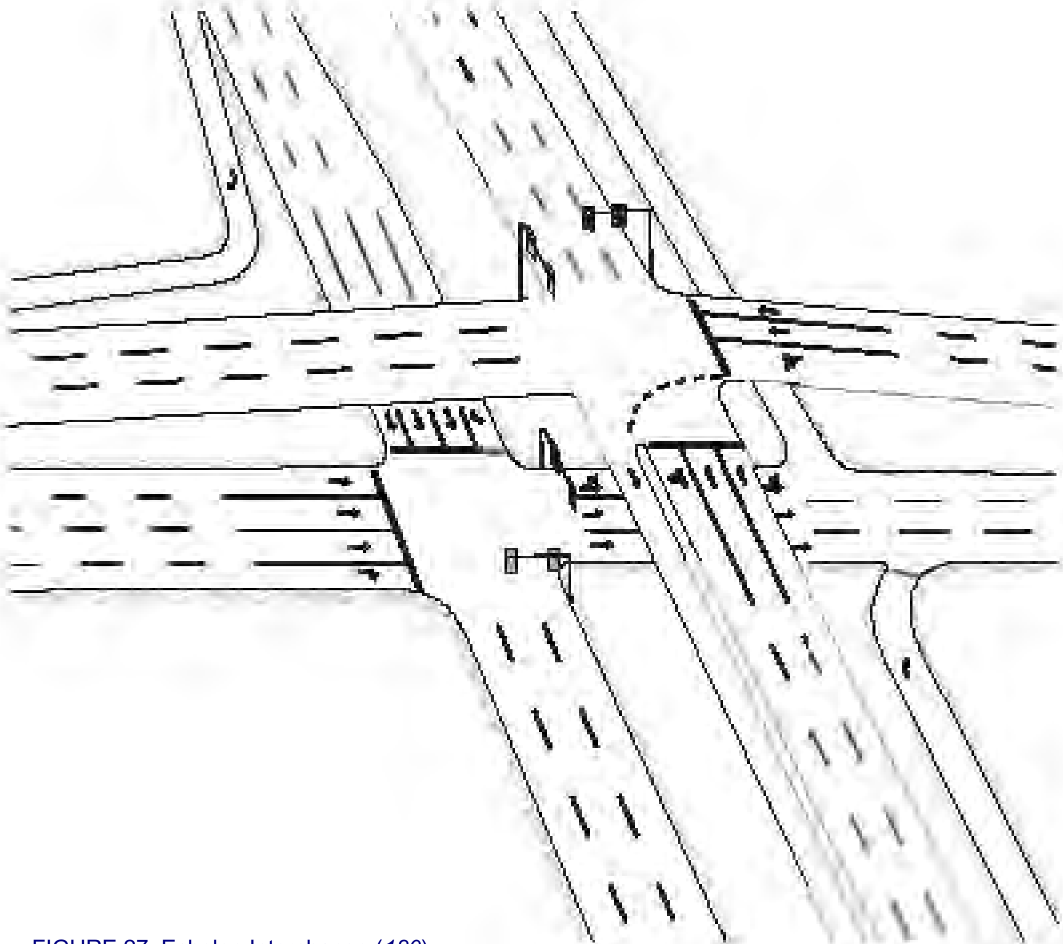


FIGURE 27 Echelon Interchange (*180*).

TABLE 44
RECOMMENDED TRAVEL TIME FOR DETERMINING SIGHT DISTANCE FOR LEFT AND RIGHT TURNS ONTO THE MAJOR ROAD AT STOP-CONTROLLED INTERSECTIONS (*181*)

Vehicle Type	Travel time (sec) at the 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 0.75 sec for trucks for each additional lane to be crossed.

TABLE 45
ISD CRITERIA FOR UNCONTROLLED INTERSECTIONS BASED ON STOPPING FROM A REDUCED SPEED (*181*)

Design Speed (km/h)	Sight Distance (m)[a]	Design Speed (mph)	Sight Distance (ft)[a]
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		

Note: Modified for consistency with the SSD model recommended by Fambro et al. (*74*).
[a]Recommended length of the leg of the clear sight triangle along each intersection roadway.

special sight-distance provisions for trucks are recommended, because it is highly unlikely that there would be substantial truck volumes at an uncontrolled intersection as traffic volumes for all vehicle types at such intersections are generally low. The sight distances are based on a PRT of 2.5 sec, rather than 2.0 sec used in current AASHTO policy.

Prior to *NCHRP Report 383*, Easa (*184*) looked at models proposed for uncontrolled intersections by McGee et al. (*185*) and Mason et al. (*186*). These models expanded the AASHTO policy to implicitly consider vehicle deceleration along an approach to an uncontrolled intersection and vehicle length. He demonstrated that there are conditions when using deceleration rates rather than stopping distances that can result in sight distances that are less than desired. The difference is greater when the difference between the design speeds of the intersecting roads is larger.

Intersections with Yield Control on the Minor Road

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; therefore, only the approach sight triangle for left- and right-turn maneuvers needs to be considered (*181*).

Intersections with Traffic Signal Control

The current AASHTO policy recommends the same sight distance at signalized intersections as are used at Stop-controlled intersections. A less restrictive policy was recommended by Harwood et al. (*181*), except at intersections where flashing signal operations or right turn on red are permitted. For these conditions, the sight distance needs are similar to Stop-controlled intersections. They encourage policies that would make sight distance a consideration in determining whether flashing operations or right-turn-on-red maneuvers should be permitted.

Left Turns from Major Road

Harwood et al. (*181*) recommended that a sight-distance policy for left turns from the major road should be presented in AASHTO policy as a separate case. It would only need to be checked for three-leg intersections on or near horizontal curves and intersections on divided roadways (roadways with medians). For other conditions (e.g., four-leg intersections on undivided roadways and three-leg intersections on tangent undivided roadways), the provision of SSD and ISD for Stop-controlled approaches ensures adequate sight distance for left turns from the major roadway. The sight distance should be based on a gap-acceptance approach to be compatible with the recommended policy for left and rights turns from Stop-controlled approaches. The critical gaps recommended are listed in Table 46. 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.

TABLE 46
RECOMMENDED TRAVEL TIMES FOR DETERMINING SIGHT DISTANCE FOR LEFT TURNS FROM THE MAJOR ROAD ACROSS OPPOSING TRAFFIC LANES (*181*)

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.

Using ISD

Harwood et al. (*181*) also recommended the following with regards to ISD:

- Measurements to determine whether specific objects within the sight triangle are sight obstructions should be based on a driver eye height of 3.54 ft (1080 mm) and an object height of 3.54 ft (1080 mm). These values should also be used in the design of vertical curves to accommodate ISD.
- Field studies showed that where necessary to obtain a sufficient view of major-road traffic, drivers on a Stop-controlled approach will move their vehicles closer to the road than the 10-ft (3-m) distance currently used in AASHTO policy. It was also found that, for most passenger vehicles, pickup trucks, and minivans, the distance from the front of the vehicle to the driver's eye is 8.0 ft (2.4 m) or less.

Sight Distance for Right-Turn Lanes

Vehicles entering a driveway using an exclusive right-turn lane may restrict the sight distance available to vehicles waiting to enter the arterial street from that driveway. The results of an analysis by Zeidan and McCoy (*187*) found that right-turn lanes much wider than normal are needed to provide adequate sight distance for driveway vehicles to enter arterial streets with design speeds from 37.3 to 62.1 mph (60 to 100 km/h). The required right-turn lane widths would typically include a channelization island between the right-turn lane and the adjacent through lane on the

TABLE 47
ACCEPTABLE GEOMETRICS FOR RIGHT-TURN LANE WIDTH WHEN A PASSENGER CAR IS THE OBSTRUCTING VEHICLE (*187*)

Design Speed mph (km/h)	Minimum Width of Right-Turn Lane,[a] ft (m)					
	Undivided Driveway [b]			Divided Driveway [c]		
	SL [d] = 0	SL = 3.3 (1)	SL = 9.8 (3)	SL = 0	SL = 3.3 (1)	SL = 9.8 (3)
37.3 (60)	19.3 (5.9)	22.6 (6.9)	29.2 (8.9)	17.4 (5.3)	20.7 (6.3)	27.2 (8.3)
43.5 (70)	19.7 (6.0)	23.0 (7.0)	29.5 (9.0)	18.0 (5.5)	21.3 (6.5)	27.9 (8.5)
49.7 (80)	20.0 (6.1)	23.3 (7.1)	29.8 (9.1)	18.7 (5.7)	22.0 (6.7)	28.5 (8.7)
55.9 (90)	20.3 (6.2)	23.6 (7.2)	30.2 (9.2)	19.3 (5.9)	22.6 (6.9)	29.2 (8.9)
62.1 (100)	20.7 (6.3)	23.9 (7.3)	30.5 (9.3)	19.7 (6.0)	23.0 (7.0)	29.5 (9.0)

Note: SL = Distance driveway vehicle stops from edge of roadway.
[a]The overall widths of right-turn lanes, which are required to provide adequate sight distances for vehicles entering the roadway from the driveway, range from 18.0 to 30.5 ft (5.5 to 9.3 m). These overall lane widths would typically include an 11.8-ft (3.6-m) right-turn lane and a 6.2- to 18.4-ft (1.9- to 5.6-m) channelization island between the right-turn lane and the adjacent through lane.
[b]Undivided driveway with 24.9-ft (7.6-m) throat.
[c]Divided driveway with 72.1-ft (22-m) overall width.
[d]Distance driveway vehicle stops from the edge of the roadway.

arterial. Table 47 lists the acceptable geometrics when a passenger car is assumed to be the obstructing vehicle. The stopping position of the driveway vehicle has a very pronounced effect on the overall width of right-lane required. Stopping positions closer to the roadway reduce the width required. However, pedestrian considerations may limit the extent to which the stopping position can be moved closer to the roadway. The authors also noted that the temporary nature of the sight restrictions caused by traffic in the right-turn lane must be considered. The volume and arrival distributions of traffic on the arterial street and using the driveway determine the probability that this sight-distance problem exists. Low traffic volumes and/or coordinated traffic flow on the arterial street may minimize the extent to which the problem occurs and the need to address it. Additional research is needed to develop volume-based warrants for the implementation of the acceptable right-turn lane geometries identified.

Sight Distance for Intersections at Other Than 90 Degrees

The following operational issues may exist when roads intersect at less than 90 degrees:

- Longer crossing distances;
- Difficulties for a driver in turning head, neck, or upper body for adequate line of sight;
- Encroachments on opposing lane during a turn; and
- Larger intersection area to accommodate turning paths of larger design vehicles.

Gattis and Low (*188*) investigated the constraints on the angle of a left-skewed intersection as affected by the vehicle body limiting a driver's line of sight to the right. This was done by measuring, in a variety of vehicle types, the maximum vision angle to the right that the vehicle allowed the driver to have. Two driving positions were considered a "sit back" and a "lean forward." A 13.5-degree vision angle (with respect to a line perpendicular to the vehicle path) was selected to represent an intermediate posture between the sit-back and the lean-forward positions. With a 13.5-degree vision angle in some restrictive vehicles, the 60-degree minimum intersection allowed in the *Green Book* will cause the driver's line of sight to be obstructed by the vehicle itself and offer only limited sight distance. When the acute angle is to the minor road driver's right, minimum angles of 70 degrees or more may be more appropriate, depending on the through road speed.

Locations where the major roadway is curved and the minor roads extend or project from the tangents create intersections with some unusual turning movements and right-of-way assignments. Gattis (*189*) determined that this intersection type needs a much larger roadside area clear of sight obstructions than that required solely by criteria for SSD. Drivers approaching curves to the right, where the minor road projects straight ahead, make what is operationally a left turn at running speed. The drivers at such intersections need sufficient sight distance to perceive oncoming cars and either stop or safely cross the left lane before the oncoming car arrives. He notes that unless a location has an accident history, the costs of remedial actions may be difficult to justify. When remedial actions are needed, possible actions to address the situation include:

- Removing obstructions to improve the line of sight for the driver (which may be impractical because a very large visually unobstructed zone would be needed).
- Providing a separate, sheltered left-turn bay for traffic on the approach where the major road curves to the right and the projecting tangential road veers to the left.
- Realigning the intersection so that the minor road will not project along a tangent from the major.
- Requiring motorists making the left turn onto projecting roads to come to a stop before turning left.

The second and third remedies might give left-turning drivers a greater sense of performing a minor movement and reinforce the need to yield. They could also encourage them to slow as they maneuver into the turn bay.

Improving Sight Distance in Urban Areas

Tipaldo (*190*) discussed practical approaches employed by the New York City DOT in improving corner sight distance at urban intersections. He notes that at several locations within New York City it is not practical (or politically feasible) to remove certain obstructions. Drivers compensate by moving their stopped vehicle to the edge of the near-side parking lane to improve visibility. Geometric-related methods used to improve corner sight distance include restricting parking, creating one-way street networks, placing fire hydrants near corners (which reduces the number of parking spaces removed due to fire hydrants and sight distance needs), using pavement markings to funnel traffic to the center of a street on wide streets, installing neckdowns or bulbs to lower operating speeds, and prohibiting parking at corners.

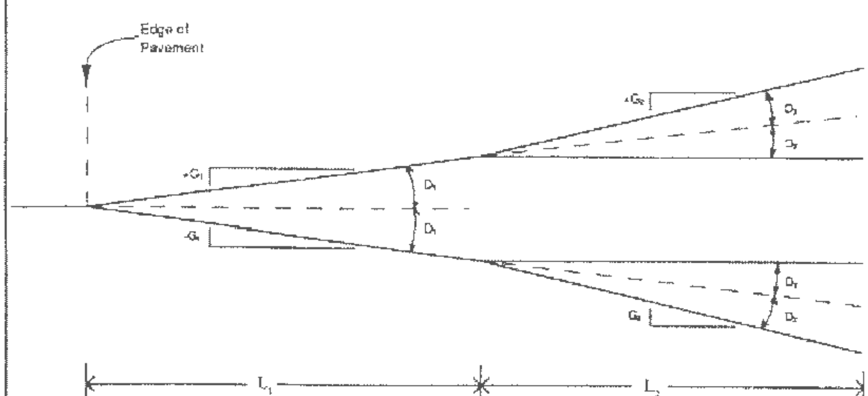
DRIVEWAY DESIGN

Williams et al. (*191*) presented vertical curve alignment guidelines for the design of driveways. Figure 28 shows their recommended guidelines, including suggested maximum grades and grade changes for various driveway classifications and volumes. They commented that steep grades inhibit the performance characteristics of vehicles, especially for braking and acceleration. Additionally, vehicle operators are often unable to judge necessary stopping and acceleration distances that are associated with steep grades. Grade changes also present operational problems if the change is so abrupt that adequate vehicular clearance is not available. Safety problems can be a concern when poor sight distances result from sharp grade changes. They note that for extreme grade changes, especially those in excess of the maximum values listed in Figure 28, vertical curves should be constructed to allow for improved operations and safety.

The PROWAAC report, "Building a True Community" (*35*), provides the following information regarding sidewalks and driveway crossings:

- Sidewalks shall contain a pedestrian access route and a reduced vibration zone.
- The minimum clear width of the pedestrian access route shall be 60 in. (1525 mm).
- The clear width of the pedestrian access route may be reduced to 48 in. (1220 mm) at driveways and alley crossings; accessible parallel parking locations with constraints, where necessary to make building entrances accessible; and at street fixtures.
- The maximum cross slope on the pedestrian access route shall be 1:48.

Eck and Kang (*192*) used computer software to develop geometric design standards to accommodate low-ground-



	G_1	G_2	D_1 or D_2	*L_1 (ft.)
High Volume (Commercial, Industrial)	±3%	±5%	±3%	40
Low Volume (Commercial, Industrial)	±5%	±8%	±6%	40
Residential	±8%	±15%	Veh. Clear.	10

* For driveways with restrictive sight distances, it is often desirable to have L_1 equal to the maximum length of the design vehicle but not less than the values indicated in the table. For certain combinations of grades where no sight distance geometric or operational constraints exist, it may be possible to justify values of L_1 that are less than those in the table.

FIGURE 28 Suggested maximum grades (*G*), maximum changes in grade (*D*), and minimum lengths of grade G_1 (L_1) (*191*).

TABLE 48
MAXIMUM GRADES AND ELEVATION DIFFERENCES FOR TYPE I PROFILE TO ACCOMMODATE THE DESIGN LOW-CLEARANCE VEHICLE (*192*)

Width of Level Section ft (m)	Maximum Safe Grades g (%)	Maximum Elevation Differences in. (mm)
4.7 (1.4)	2.5	9.0 (228.6)
6 (1.8)	2.7	9.5 (241.3)
8 (2.4)	2.9	9.7 (246.4)
10 (3.1)	3.1	10.2 (259.1)
12 (3.7)	3.3	10.4 (264.2)
14 (4.3)	3.5	10.6 (269.2)
16 (4.9)	3.8	11.1 (281.9)
18 (5.5)	4.1	11.5 (292.1)
20 (6.1)	4.4	11.8 (299.7)
22 (6.7) or more	4.6	11.8 (299.7)

Note: Type I profile = upgrade (+g), followed by a level portion of length "W," and then followed by a downgrade (–g).

TABLE 49
MINIMUM CREST VERTICAL CURVE LENGTH FOR TYPE II PROFILES TO ACCOMMODATE DESIGN LOW-CLEARANCE VEHICLE (*192*)

Algebraic Difference (%)	Curve Length ft (m)
1	4 (1.2)
2	8 (2.4)
3	12 (3.7)
4	16 (4.9)
5	20 (6.1)
6	24 (7.3)
7	28 (8.5)
8	32 (9.8)
9	35 (10.7)
10	39 (11.9)

Note: Type II profile = upgrade (+g) followed by a downgrade (–g).

clearance vehicles. The HANGUP software package was used to evaluate some of the low-clearance design standards currently in use. Assuming a 36-ft (11.0-m) wheelbase and 5 in. (127.0 mm) of ground clearance, the authors concluded

- The AREA design standard that was used in the 1990 *Green Book* (*1*) for grade crossing profiles was found to accommodate most low-clearance vehicles.
- The ITE-recommended practice on driveway design (*193*) was found to not accommodate low-clearance vehicles. Using a maximum grade change of 3 percent, the authors found that the standard low-clearance vehicle will have problems with the design.

Values considered as the dimensions for the design of low-ground-clearance vehicles were determined to be greater than or equal to 5 in. (127.0 mm) of ground clearance and wheelbases of less than or equal to 36 ft (11.0 m). Table 48 presents the maximum grades and elevation differences for a Type I profile [described as an upgrade (+g), followed by a level portion of length "W," and then followed by a downgrade (−g)]. Table 49 presents the minimum crest vertical curve lengths for Type II profiles [described as an upgrade (+g), followed by a downgrade (−g) that can safely accommodate the design low-clearance vehicle]. Table 50 presents the minimum lengths of sag vertical curves to prevent overhand dragging for the design of low-clearance vehicles. For example, when a low-ground-clearance vehicle whose rear overhand length is 5 ft (1.5 m) and ground clearance is 5 in. (127.0 mm) traverses a sag vertical curve with a 10 percent algebraic difference in grade, the minimum length of the sag curve should be 3 ft (0.9 m). The authors note that there is still considerable work to be done before criteria to accommodate low-clearance vehicles becomes a formal part of any design policy. For example, physical characteristics of low-clearance vehicles need to be collected from all regions of the country.

CORNER CLEARANCE

Long and Gan (*194*) developed a model to calculate minimum corner clearances (MCC). Corner clearance was defined as the distance from an intersection to a driveway,

TABLE 50
MINIMUM LENGTHS OF SAG VERTICAL CURVE LENGTHS TO PREVENT OVERHAND DRAGGING FOR DESIGN LOW-CLEARANCE VEHICLE (*192*)

Overhang Length (ft)	Minimum Lengths of Sag Vertical Curves (ft)								
	1	2	3	4	5	6	7	8	9
1	0.6	0.3	0.2	0.2	0.1	0.1	0.1	0.1	0.1
2	2.4	1.2	0.8	0.6	0.5	0.4	0.4	0.3	0.6
3	5.4	2.7	1.8	1.4	1.1	0.9	0.8	0.7	0.6
4	9.6	4.8	3.2	2.4	1.9	1.6	1.4	1.2	1.1
5	14.9	7.5	5.0	3.8	3.0	2.5	2.2	1.9	1.7
6	21.5	10.8	7.2	5.4	4.3	3.6	3.1	2.7	2.4
7	29.3	14.7	9.8	7.3	5.9	4.9	4.2	3.7	3.3
8	38.3	19.1	12.8	9.6	7.7	6.4	5.5	4.8	4.3
9	48.4	24.2	16.2	12.1	9.7	8.1	6.9	6.1	5.4
10	59.8	29.9	19.9	15.0	12.0	10.0	8.6	7.5	6.7

Note: Clearance values are in inches. Grade difference + 10%; 1 ft = 0.305 m; 1 in. = 25.4 mm.

measured from the closest edge of the pavement of the intersection road to the closest edge of the pavement of the driveway. The model produces a refined MCC by applying a set of adjustment factors to an initial MCC. The authors argued that existing tabular guidelines are rigid, whereas their model has the potential to better meet the level of flexibility needed in access management. Of the nine adjustment factors provided, three were designed to reproduce the MCCs of existing guidelines, and the others were included to give the flexibility that is not found in existing guidelines. The authors comment that further studies are needed to refine the suggested adjustment factors and that different factors are needed for approach and departure corner clearances. The model is shown in Table 51. The authors also discussed a check within $MCC_{(saturation\ flow)}$ calculations on the assumption of whether trailing vehicles catch up and close the empty spaces left by queue dissipation. How the check is to be used within the series of equations and adjustment factors listed in Table 51 was not clear in the paper.

Gluck et al. (*15*) reviewed corner clearance criteria in *NCHRP Report 420*. They reviewed the procedure developed by Long and Gan (*194*) and commented that there is little basis for assessing the various adjustment factors, the validity of the basic models, and the practicality of the results. The procedure also does not consider queuing that would decrease as the green per cycle increases, and it focuses on establishing spacing guidelines for corner clearance—not assessing effects. Gluck et al. (*15*) assembled corner clearance criteria for selected cities, counties, and states. They noted that there is a wide range of practices, with values ranging from 16 ft (4.9 m) (urban area in Iowa) to more than 300 ft (91.5 m) (Colorado). Many fall within the 100- to 200-ft (30.5- to 61.0-m) range. They used case studies of corner clearances to illustrate current practices, problems, and opportunities. They developed application guidelines as part of *NCHRP Report 420*. They note that from a planning perspective, two actions should be encouraged; both require a proactive approach to corner clearances.

- Establishing the desirable location of access points before property is subdivided or developed and
- Establishing minimum requirements for property frontages in zoning and subdivision regulations.

The principles that should guide corner clearance and driveway planning are listed here. Actions vary for retrofits and new facilities.

- Ideally, no driveways should be permitted on major highways. This requires safe and convenient alternative access and reasonable internal site circulation.
- Where this is not possible, major highways should have physical (restrictive) medians to preclude left turns. Each corner parcel should have one driveway per roadway that is placed as far from the intersections as possible.
- Along undivided major highways, it is desirable to eliminate left-turn ingress and egress at driveways within the "influence area" of an intersection. This may entail providing short sections of a median divider and/or adopting a driveway design that discourages or prevents left-turn maneuvers.
- Driveways should be located as far from the intersection as possible—either at or within 10 ft (3.1 m) of the property line farthest from the intersection.

Potential corrective retrofit actions are listed here.

- Locating driveways at the farthest edge of the property line from the intersection.
- Consolidating driveways with adjacent properties, thereby increasing corner clearances.
- Closing driveways along the arterial and requiring property access from the secondary road.
- Installing a raised median barrier on approaches to intersections to preclude left turns into or out of a driveway.

TABLE 51
PROCEDURE TO CALCULATE MINIMUM CORNER CLEARANCE (*194*)

The longer of two minimum distances (MCC_{SF} or MCC_{UF}) should govern the minimum standards for corner clearances.	
$MCC_i = IMCC_i \times f_{ft} \times f_{mt} \times f_{dc} \times f_{dw} \times f_{dv} \times f_{hv} \times f_{pv} \times f_{ts} \times f_{tw}$	
Where MCC_i = MCC (m) under traffic flow condition i, where i is either saturated flow (SF) or understatured flow (UF); $IMCC_i$ = IMCC distance (m) for traffic flow condition i; f_{ft} = adjustment factor for facility type major arterial (1.00), minor arterial (0.90), major collector (0.80), minor collector (0.70); f_{mt} = adjustment factor for median type nonrestrictive (1.00), restrictive (0.50); f_{dc} = adjustment factor for driveway channelization two-way, full access (1.00), right in or right out only (0.85); f_{dw} = adjustment factor for driveway width two-way, full access: <9 m (1.10), 9 to 12 m (1.00), >12 m (1.10) right in or right out only: <5 m (1.10), 5 to 7 m (1.00), >7 m (1.10);	f_{dv} = adjustment factor for driveway traffic volume ≥ 1,500 vpd (1.2), 1,201–1,500 (1.10), 901–1,200 (1.00), 601–900 (0.95), 301–600 (0.90) ≤ 300 (0.85); f_{hv} = adjustment factor for peak-hour driveway heavy vehicle volume ≥ 80% (1.50), 61–80 (1.30), 41–60 (1.15), 21–40 (1.05) ≤ 20 (1.00); f_{pv} = adjustment factor for coincidence of driveway and arterial peak period volumes ≥ 80% (1.20), 61–80 (1.10),41–60 (1.00), 21–40 (0.90) ≤ 20 (0.80); f_{ts} = adjustment factor for driveway corner turning speed for design or representative vehicles <16 km/h (1.00) ≥ 16 km/h, or curb return radius ≥ 5 m (0.95); and f_{tw} = adjustment factor for curb lane width 3.0 m (1.10), 3.3 (1.05), 3.6 (1.00), 3.9 (0.95), 4.2 (0.90).
$IMCC_{SF} = [(k \times c + g - \Sigma h_i) \times s] / c$	
where: g = effective green time (sec); h_i = intersection entering headway for i^{th} vehicle (sec); c = entering headway for steady flow (or saturation headway) (sec); s = space per vehicle stopped in queue, usually assumed to be 7.63 m (25 ft); and k = number of vehicles (the first k vehicles) with variable entering headways.	Thus, by using the values suggested in *HCM* and assuming that the first four entering headways are 4.0, 2.8, 2.5, and 2.2 sec, respectively, the saturation headway is 2 sec, and the space per vehicle is 7.63 m (25 ft), the $IMCC_{SF}$ for an approach with a 40-sec effective green time can be computed as follows: $IMCC_{SF} = [(4 \times 2 + 40 - \{4.0 + 2.8 + 2.5 + 2.2\}) \times 7.63] / 2 = 139$ m (456 ft)
$IMCC_{UF}$ = values (ft) listed below[a]	

Speed (mph)	30	35	40	45	50	55	60
Desirable[b]	325[d]	425	525	630	750	875	1005
Limiting[c]	215	270	335	405	480	565	655

Note: 1 ft = 0.305 m; 1 mi = 1.61 km.
[a]All values were originally computed in feet and were rounded to the nearest 5 ft.
[b]2.5 sec perception–reaction time; 3.5 ft/sec^2 average deceleration while moving laterally into turn bay and an average 6 ft/sec^2 deceleration thereafter; 10 mph speed differential.
[c]1.0 sec perception–reaction time; 4.5 ft/sec^2 deceleration while moving laterally into turn bay and an average 9.0 ft/sec^2 thereafter; 10 mph speed differential.
[d]Distance equals deceleration distance (distance to decelerate from speed to a stop while maneuvering laterally into a turn bay) plus distance traveled in perception–reaction time.

LEFT-TURN LANES

A previous NCHRP Synthesis (*195*) provided information on left-turn treatments at intersections. It covered the following subjects including guidelines on the need for a left-turn lane, traffic studies and design considerations, signing and pavement markings, traffic signal needs, and performance measures.

Guidelines for Left-Turn Lanes

Pline (*195*) reported in the synthesis that the majority of states (72 percent) and local jurisdictions (62 percent) indicated that they use the 1990 *Green Book* (*1*) for determining left-turn lane requirements. The *Green Book* approach is based on Harmelink's (*196*) graphs. Harmelink's method was expanded in 1990 by ITE Committee 4A-22 (*197*) to provide guidelines for 30 mph operating speeds, two-lane roadways, four-lane undivided roadways, and four-lane divided roadways.

The Idaho Transportation Department (*198*) developed the rural left-turn warrant graph (Figure 29) as a basic consideration for left-turn lanes with an engineering study required to analyze operating speeds, traffic volumes, sight distance, passing opportunities, number of anticipated turning movements, and accident history. Modur et al. (*199*) at the University of Texas at Austin developed criteria for selection of a left-turn median design as shown in Figure 30. Accidents are the major special consideration other than traffic volumes to justify a left-turn lane. Generally, three

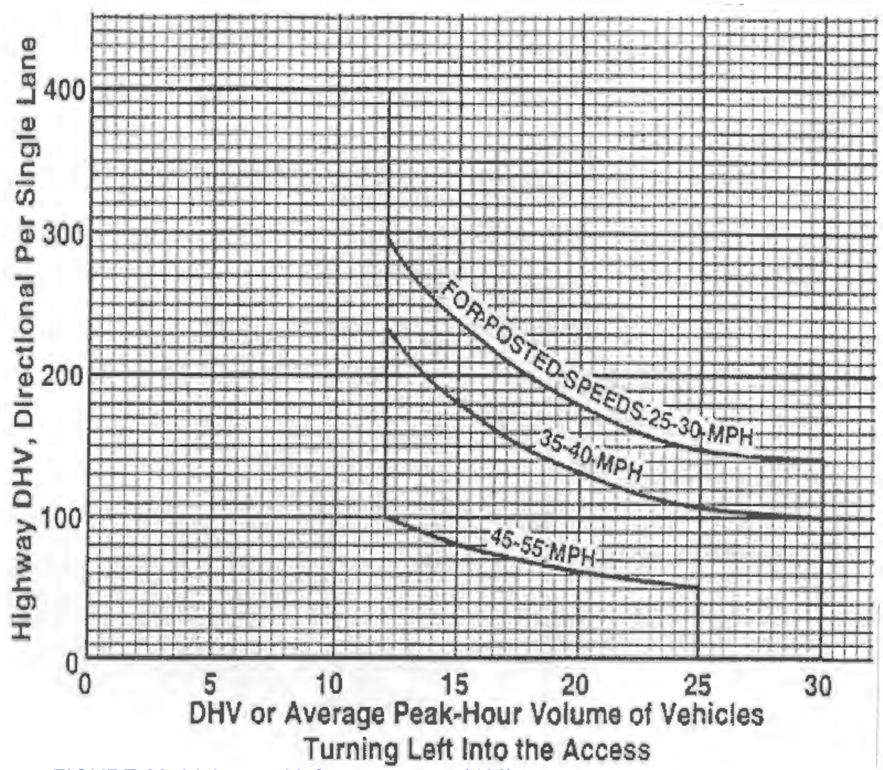


FIGURE 29 Idaho rural left-turn warrant (*198*).

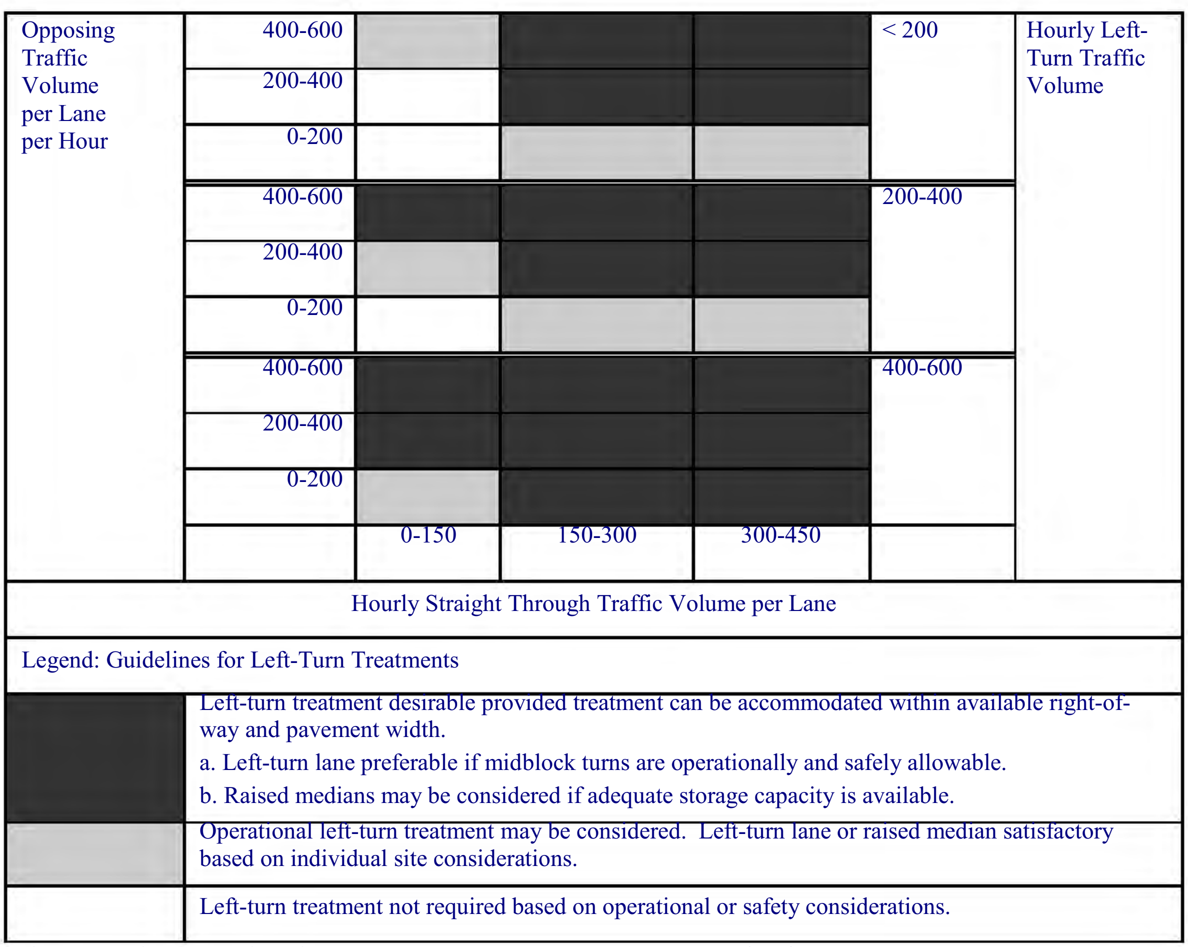


FIGURE 30 Guidelines for left-turn treatment, single isolated intersection (*199*).

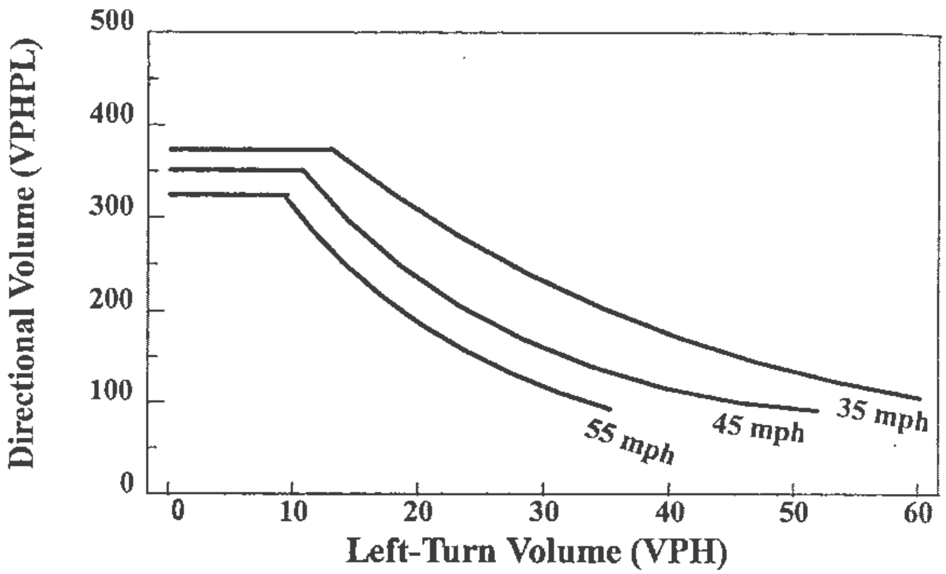


FIGURE 31 TTI left-turn guidelines (*207*).

or four left-turn accidents per year per direction (not per intersection) are the critical number to justify a separate left-turn lane.

Koepke (*200*) provided a summary of several approaches used to determine when separate turn lanes should be required. He presented the guidelines from the 1985 *HCM Special Report* (*201*) and an FHWA report for left-turn lanes at signalized intersections (*202*); the 1985 *HCM* (*201*) for right-turn lanes at signalized intersections; *NCHRP Report 348* (*14*), the 1990 *Green Book* (*1*), Harmelink (*196*), and ITE work that built on Harmelink (*197*), and a study done in Illinois (*203*) for left-turn lanes at unsignalized intersections; and the Colorado and Virginia DOT guidelines (*204,205*) for right-turn lanes at unsignalized intersections. In addition, he presented guidelines based on accidents that were developed in Kentucky (*206*).

Several existing guidelines on when to install a left-turn lane are based on turn volume. Hawley and Stover (*207*) proposed guidelines that are based on delay of the through volume traveling in the same direction as the left-turning vehicles. The guidelines were developed for nonsignalized intersections along four-lane, undivided arterial streets with nonplatoon flow characteristics. The TEXAS Model for Intersection Traffic simulation was used to calculate delays incurred due to left-turning vehicles. For each set of simulation runs, the speed and through volume were held constant while the left-turn volume increased. The goal was to identify the left-turn volume where a sharp increase in delay was recorded. The minimum delay was defined as the delay prior to the sharp increase. The best-fit exponential lines are shown in Figure 31. These lines were modified with consideration of not presenting an undue accident risk. A conflict analysis was performed based on the premise of determining the probability of two vehicles (a left-turning vehicle immediately followed by a through vehicle) arriving at the intersection simultaneously in the left lane. The probability of this occurring would be the probability of a potential conflict at the intersection due to the lack of a left-turn bay. A probability of 0.01 was assumed. The horizontal lines on the end of each of the curves accounts for the results of the conflict analysis. In Figure 31, left-turn lanes are recommended for traffic volumes above and to the right of the curve corresponding to the speed of the facility. The essence of the guidelines could be summarized with the following statements:

- With low directional volumes, the left lane will function as a pseudo left-turn lane and not impact the through traffic traveling in the right lane; however,
- With higher directional volumes, the introduction of relatively few left-turn vehicles will result in a substantial increase in delay to the through vehicle.

Left-Turn Lane Lengths

Koepke (*200*) provided a summary of how to determine the length of a turn lane based primarily on information from Harmelink (*196*) and the 1990 *Green Book* (*1*).

Kikuchi et al. (*208*) recommended lengths for left-turn lanes at signalized intersections. They analyzed lane lengths from two aspects: (1) the probability of overflow of vehicles from the turning lane and (2) the probability of blockage of the entrance to the turning lane by the queue of vehicles in the adjacent through lane. The following were considered in developing the recommendation: signal timing, left-turn volume, through volume, threshold probabilities and through field surveys, turning maneuver time, and space requirement per vehicle on the lane. The authors model the fluctuation of the number of left turns in the queue at the beginning of each protected green phase as a

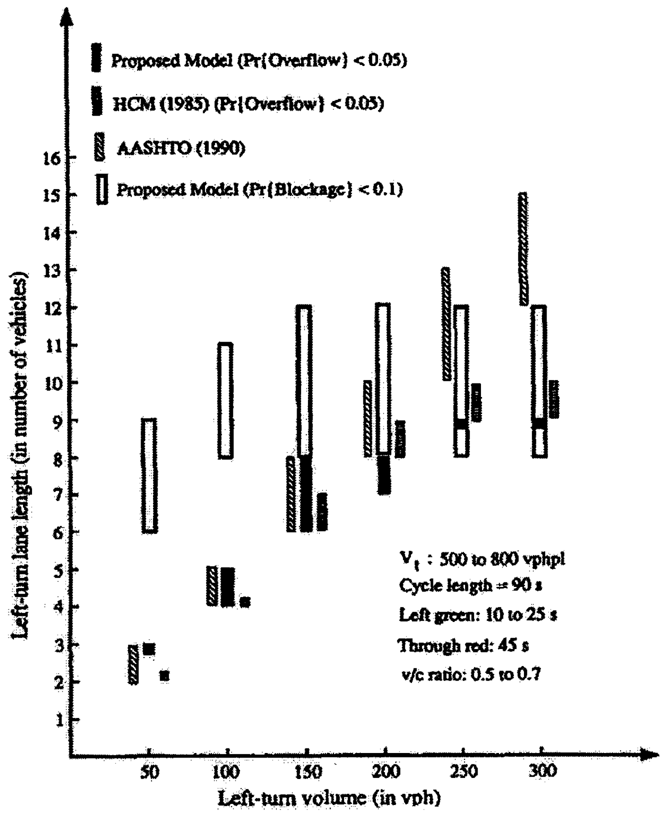


FIGURE 32 Comparison of proposed model with existing guidelines (*208*).

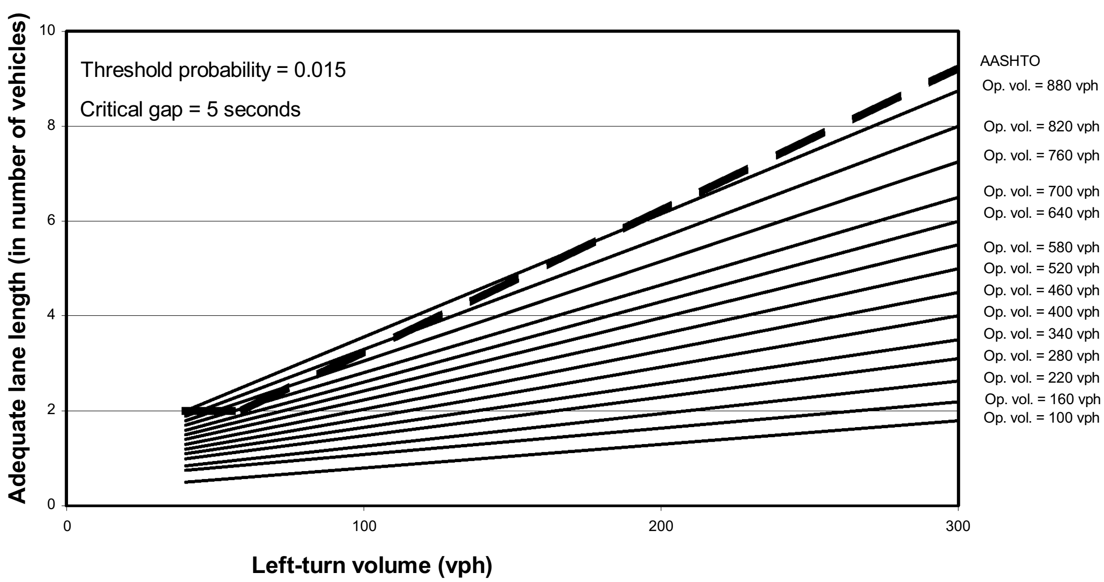


FIGURE 33 Comparison of AASHTO and proposed model lane lengths (*209*).
Note: The solid lines represent the results of the proposed model. For practical applications, it is recommended to use a minimum of two vehicle lengths.

Markov process. The comparison of the proposed lengths with the 1990 *Green Book* (*1*) and 1985 *HCM Special Report* (*201*) is shown in Figure 32. The *Green Book* and *HCM* guidelines are derived from the standpoint of preventing overflow of the left-turn lane only. The *Green Book* recommends lengths that are longer than the *HCM* or the Kikuchi et al. (*208*) model, with the difference increasing as the left-turn lane volume increases. When lane blockage is

TABLE 52
THE 50th, 85th, AND 95th PERCENTILE STORAGE LENGTH (VEHICLE UNITS) FOR MOVEMENTS FOR WHICH NO OPPOSING VEHICLES ARE INVOLVED (*210*).

Separate Phase Lane Volume vph	Percentile Value	Cycle Length = 60 sec: Effective Green Time						
		10	15	20	25	30	35	40
50	50th	1	0	0	0	0	0	0
	85th	2	1	1	1	1	1	1
	95th	2	2	2	2	2	2	2
100	50th	1	1	1	1	1	1	0
	85th	3	2	2	2	2	2	1
	95th	4	3	3	3	3	3	2
150	50th	2	2	2	1	1	1	1
	85th	4	3	3	3	2	2	2
	95th	6	4	4	4	3	3	3
200	50th	4	2	2	2	2	1	1
	85th	9	4	4	4	3	3	2
	95th	13	5	5	4	4	3	3
250	50th	∞	3	3	2	2	2	1
	85th	∞	6	5	4	4	3	3
	95th	∞	8	6	5	5	4	4
300	50th		5	3	3	2	2	2
	85th		10	6	5	4	4	3
	95th		14	7	6	5	5	4
350	50th		32	4	3	3	2	2
	85th		∞	7	5	5	4	3
	95th		∞	9	7	6	5	5
400	50th		∞	5	4	3	3	2
	85th		∞	9	6	5	5	4
	95th		∞	12	8	7	6	5
450	50th			11	5	4	3	2
	85th			21	7	6	5	4
	95th			27	10	8	6	5
500	50th			∞	6	4	3	3
	85th			∞	10	7	6	5
	95th			∞	13	9	7	6
550	50th				9	5	4	3
	85th				16	8	6	5
	95th				23	10	8	6
600	50th				∞	6	4	3
	85th				∞	10	7	6
	95th				∞	13	9	7
650	50th					8	5	4
	85th					15	8	6
	95th					19	10	7
700	50th					19	6	4
	85th					43	9	6
	95th					55	12	8
750	50th					∞	7	4
	85th					∞	13	7
	95th					∞	19	10
800	50th						12	5
	85th						25	9
	95th						33	12

considered, the values recommended by Kikuchi et al. (*208*) are considerably longer than the *Green Book* or the *HCM.* The comparison suggests that for a small left-turn volume, attention should be paid to the possibility of lane blockage, whereas for a large left-turn volume, attention should be given to the possibility of lane overflow.

Chakroborty et al. (*209*) developed recommended left-turn lane lengths for unsignalized intersections using a mathematical model that considers the volume of turning vehicles, volume of opposing vehicles, critical gap, threshold probability (probability that a given length will result in overflows), and vehicle mix. The model produces the number of vehicles, this was then converted to lane length using field studies of the amount of space consumed by different vehicle types. The validity of the model was checked by computer simulation and compared to the lengths suggested by AASHTO (Figure 33). This study provided a method for computing left-turn lane length based not only on the left-turning volume, but also on critical gap, opposing volume, and vehicle mix.

Oppenlander and Oppenlander (*210*) used a stochastic simulation model to generate probability distributions for queue lengths. These queue lengths are then used to determine storage requirements for approach lanes at signalized intersections. They produced a series of tables that provided 50th, 85th, and 95th percentile storage lengths (in vehicle units) for different combinations of cycle lengths, effective green time, and lane volumes. The values are applicable for any movement for which no opposing vehicles are involved. Table 52 is an example of one of the tables generated, and Table 53 provides an example of the use of the procedure.

Offset Left Turns

The AASHTO *Green Book* recognizes the reduction of sight distance at opposing left-turn lanes and suggests the use of parallel or tapered offsets as a means of improving sight distance. However, the *Green Book* does not give specific guidelines as to what offsets are required.

In 1992, McCoy et al. (*211*) developed guidelines for offsetting opposing left-turn lanes at 90-degree intersections on level, tangent sections of four-lane divided roadways with 12-ft (3.7-m) lanes (Table 54). The authors note that the guidelines should not be used for situations outside the scope of these limitations, such as at skewed intersections or intersections on horizontal curves. Figure 34 illustrates negative and positive offsets. The vehicle positioning used to develop the guidelines was determined from the observations of vehicles making left turns from 12-ft (3.7-m) left-turn lanes in 16-ft (4.9-m) curbed medians with 4-ft (1.2-m) medial separators at signalized intersections on four-lane divided roadways. [The minimum offsets were calculated using 8.5 sec as the time needed to complete a left turn. The 8.5-sec value is greater than the times recommended as part of *NCHRP Report 383* (*181*) (Table 45). Therefore, the minimum offsets less than those presented in Table 54 that would result in the gap values listed in Table 45 were used.

TABLE 53
EXAMPLE OF THE USE OF OPPENLANDER PROCEDURE (*210*)

Lane Group on East Approach	Lane Volume (vph)	Green Time (sec)	85th Percentile Storage Length Vehicle unit	ft (m)	Design Length ft (m)
LT	110	12	4	100 (30.5)	
TH	425	34	8	200 (61.0)	200 (61.0)
TH/RT	425	34	8	200 (61.0)	

Notes: Given four-leg intersection, 75-sec cycle length; three-phase signal operation, assume 25 ft (7.6 m) vehicle length.
LT = left turn; TH = through; RT = right turn.

TABLE 54
OFFSET GUIDELINES FOR PASSENGER CARS (*211*)

Design Speed (mph)	Minimum Offsets,[a] ft		Desirable Offsets,[b] ft	
	Passenger Car[c]	Truck[d]	Passenger Car[c]	Truck[d]
40	1.0	2.5	2.0	3.5
45	1.0	3.0	2.0	3.5
50	1.5	3.0	2.0	3.5
55	1.5	3.0	2.0	3.5
60	1.5	3.0	2.0	3.5
65	1.5	3.0	2.0	3.5
70	1.5	3.0	2.0	3.5

Notes: Findings are for 90-degree intersections on level, tangent sections of four-lane divided roadways with 12-ft lanes. Offset is the lateral distance between the left edge of a left-turn lane and the right edge of the opposing left-turn lane. If the right edge of the opposing left-turn lane is to the left of the left edge of the left-turn lane, the offset is a negative offset. If it is to the right, it is a positive offset.
1 ft = 0.305 m.

[a]The minimum offsets are those required to provide the opposing left-turn vehicles with the required sight distances.
[b]The desirable offsets are those that provide the opposing left-turn vehicles with unrestricted sight distance.
[c]Opposing left-turn vehicle is a passenger car.
[d]Opposing left-turn vehicle is a truck.

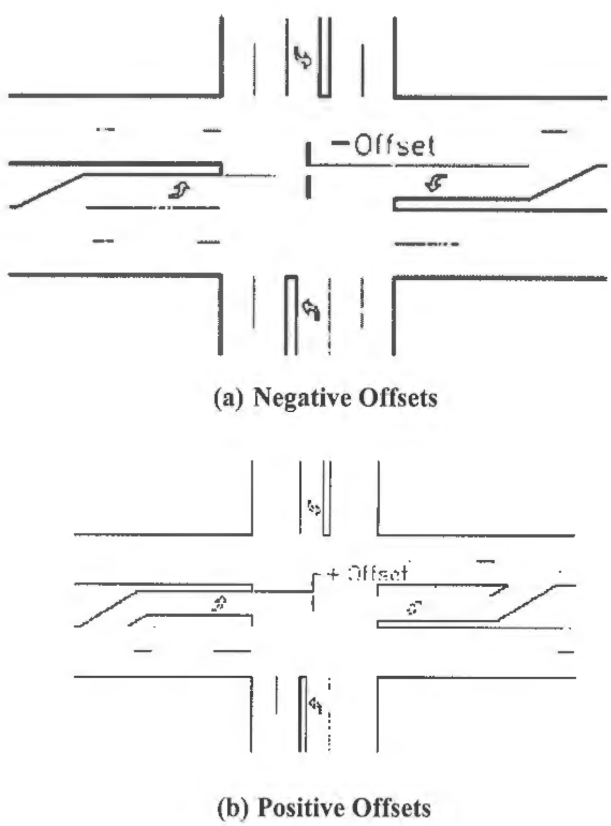


FIGURE 34 Negative and positive offsets between opposing left-turns lanes (*211*).

Several years later, Tarawneh and McCoy (*212*) presented guidelines for offsetting opposing left-turn lanes developed based on observations of driver behavior over a wider range of offset conditions. The guidelines were based on left-turning vehicle positioning and maneuver-time data collected at intersections with left-turn lane offsets ranging from −14 to 6 ft (−4.3 to 1.8 m). These guidelines (listed in Table 55) include the case in which both the left-turning and the opposing left-turning vehicles are unpositioned. This case requires larger offsets and should be considered when there are high percentages of older drivers. The developed guidelines used a required sight distance calculated using the 95th percentile maneuver time found during the study. The study sites were opposing left-turn lanes on level, tangent approaches at four-leg, 90-degree, signalized intersections on urban arterial streets. The posted speed limit at all four locations was 35 mph (56.4 km/h). All of the left-turn maneuvers observed were made during permitted left-turn phases. The offsets and number of opposing lanes varied among the four sites.

A study by Tarawneh and McCoy (*213*) on the effects of the offset between opposing left-turn lanes found that driver performance is affected by the offset present. Left-turn performance of 100 subjects within 3 age groups was evaluated under normal driving conditions at 4 intersections of different left-turn offset configurations. When large negative offsets exist (i.e., the turning vehicle must cross more pavement during the turn and more of the opposing left-

TABLE 55
GUIDELINES FOR OFFSETTING OPPOSING LEFT-TURN LANES ON DIVIDED ROADWAYS (*212*)

Opposing Left-Turn Vehicle		Minimum Offset,[a] m							Desirable Offset,[b] m
		Design Speed, km/h							
Type	Location[c]	50	60	70	80	90	100	110	
Passenger Car	Unpositioned	1.0	1.0	1.1	1.1	1.1	1.2	1.2	1.3
	Positioned	0.2	0.3	0.3	0.4	0.4	0.4	0.4	0.6
Truck	Unpositioned	1.5	1.5	1.5	1.6	1.6	1.6	1.6	1.7
	Positioned	0.8	0.8	0.9	0.9	0.9	1.0	1.0	1.1

Notes: Findings are for level, tangent approaches at four-leg, 90-degree, signalized intersections on urban arterial streets.
Offset is the lateral distance between the left edge of a left-turn lane and the right edge of the opposing left-turn lane. If the right edge of the opposing left-turn lane is to the left of the left edge of the left-turn lane, the offset is a negative offset. If it is to the right, it is a positive offset.
1 m = 3.3 ft; 1 km = 0.6 mi.

[a]The minimum offsets are those required to provide the opposing left-turn vehicles with the required sight distances.
[b]The desirable offsets are those that provide the opposing left-turn vehicles with unrestricted sight distance.
[c]Unpositioned vehicles remain behind the stop line. Positioned vehicles enter the intersection in an attempt to see past the opposing left-turning vehicle.

turn lane is in the driver's view), the size of the critical gaps for drivers turning left is higher. The authors stated that the large negative offsets may be particularly troublesome for older drivers and women drivers, who were less likely to position their vehicles within the intersection to see beyond vehicles in the opposing left-turn lane.

Other authors, for example, Joshua and Saka (*214*) in 1992, have also noted that offsetting the opposing left-turn lanes can improve the available sight distance for a left-turning motorist on a major approach. McCoy et al. (*215*) are investigating improving opposing left-turn lane sight distance by means of pavement markings. The objective of the study is to eliminate the sight distance problem by using pavement markings to encourage left-turn drivers to position themselves laterally closer to the median, thus improving their view of opposing through traffic. A before and after study is planned.

Triple Left-Turn Lanes

Ackeret (*216*) developed criteria for the geometric design of triple left-turn lanes using engineering judgment and two sites in Las Vegas, Nevada. Triple left-turn lanes are to be considered when the left-turning traffic exceeds 600 vph. He notes that the developed criteria "should not be considered an exhaustive list, but as a reference guide for the design of these new and unique facilities." He recommends that the geometric design elements for the development of triple left-turn lanes follow the general standards of practice for dual left-turn lanes. The following list is a summary of his comments regarding the geometric design of triple left-turn lanes:

- Selection of Design Vehicle and Turning Path—The triple left-turn lane geometrics should be designed either for (1) single-unit trucks or buses or (2) WB-50 design vehicles, depending on the probability of semi-trailer trucks. The design vehicles should be placed three abreast and tracked through the left-turn movements.
- Clearance—The lateral clearance between the running design vehicles should be maintained with a minimum clearance of 2 ft (0.6 m) on each side of the design vehicle overhand limits within the turning maneuver. The author recommends that the center left-turn lane increases in width to accommodate off-tracking of the design vehicle turning characteristics. The wider center lane reduces the potential for vehicle sideswipes by passenger vehicles when turning through the intersection. Under conditions of concurrent opposing left-turns within the intersection, a recommended 10 ft (3.1 m) of lateral vehicle body clearance between opposing vehicles was found to be an acceptable design criterion.
- Approach and Departure Lane Widths—On both approach and departure the minimum is 11 ft (3.4 m) with a desirable width of 12 ft (3.7 m). A key factor controlling the geometry of the downstream receiving throat width is the tracking path of the design vehicle as it transitions from a circular to a tangential motion. The width of the clear portion of the intersection may need to be widened based on the design vehicle turning characteristics.
- Median—A 2-ft (0.6-m) offset from the vehicle turning path for the vehicle in the lane closest to the median (called Lane 1) has been used in locating the median island nose. A raised median island has been found to provide a driver in Lane 1 with a visual point of reference to guide a vehicle through the left-turn maneuver. A raised median island also provides delineation for the stop bar location on the receiving street.
- Storage Bay and Taper Lengths—Because information is not available on triple left-turn lane utilization and capacity, the author recommends that the same procedures used for dual left-turn lanes be used.
- Roadway Delineation and Signage—The author comments that advance overhead signs and the

placement of pavement delineation through an intersection is critical for the effective and safe use of the triple left-turn lanes.

- Signal Design—To provide left-turn signal faces over each turning lane, special mast arm and signal pole equipment with cantilever mast arm lengths of 60 to 70 ft (18.3 to 21.4 m) have been required. These designs resulted in special mast arm, pole, and foundation designs in Las Vegas. Using a signal bridge spanning the entire intersection was considered but not accepted due to aesthetic concerns of the community.

Triple left-turn lane facilities have been considered not to be appropriate for installation at a signalization intersection when

- There is a potential for a high number of pedestrian/vehicle conflicts.
- Left-turning vehicles are not anticipated to queue evenly within the provided left-turn storage lanes due to downstream conditions (this may occur when a high potential for downstream weaving exists).
- Conditions exist that obscure or result in confusing pavement channelization markings within the intersection.
- Right-of-way restrictions prohibit adequate design vehicle turning maneuver space within the intersection.
- The installation is not economically justified when compared with other alternatives to improve intersection capacity.

Ackeret notes that additional research should be conducted to include topics such as

- Accident rate comparisons between dual and triple left-turn lane installations;
- Lane utilization;
- Left-turn lane adjustment factors;
- Determination of saturation flow rates for triple left-turn lanes;
- A comparison of left-turn capacity among single, double, and triple turn lanes; and
- The effects of downstream weaving on the uniform loading of triple left-turn storage bays and intersection left-turn capacity.

A follow-up study (*217*) evaluated sideswipe crashes at triple and dual left-turn lanes in Las Vegas. The authors compared four intersections with triple left-turn lanes with four intersections with dual left-turn lanes. Crash data were gathered within 100 ft (30.5 m) of the centerline street intersection. They found a difference in the number of crashes between triple and dual left-turn lanes; however, they argued that triple left-turn lanes would have crash rates reasonably similar to those of dual left-turn lanes if the turn lanes are designed with adequate left-turn geometry for appropriate design vehicles and provisions are made for eliminating far-side bottlenecks such as the placement of bus stops. They concluded that engineering concerns over installing triple left-turn lanes based solely on potential vehicle sideswipe crashes do not appear to be warranted. They also encouraged a more detailed analysis to identify specific factors related to design and operations needed to be considered when evaluating safety at such facilities.

RIGHT-TURN LANES

Tarawneh and McCoy (*218*) used a field investigation to study the effects of the geometries of right-turn lanes on the turning performance of drivers with respect to driver age and gender. Right-turn performance of 100 subjects within three age groups (25–45, 65–74, and 75+ years old) was evaluated under normal driving conditions at four intersections of different right-turn lane channelization and skew. Their findings included the following:

- Right-turn channelization affects the speed at which drivers make right turns and the likelihood that they will stop before making a right turn on red.
- Drivers, especially those of middle-age (25–45 years old), turn right at speeds 3.1 to 5.0 mph (5 to 8 km/h) higher on intersection approaches with channelized right-turn lanes than they do on approaches with unchannelized right-turn lanes.
- Drivers are much less likely to stop before making a right turn on red on approaches with channelized right-turn lanes.
- Drivers are less likely to attempt to make a right turn on red at a skewed intersection where the viewing angle to traffic from the left on the cross street is greater than 90 degrees.
- Drivers at skewed intersections are more likely to use their side mirrors than they are when making a right turn at red at nonskewed intersections.

Limitations of the study as noted by the authors included using volunteer subjects, being conducted only during off-peak hours, including a researcher in the vehicle with the subject, and having only one intersection represent a geometric feature.

McCoy et al. (*219*) developed guidelines for the use of right-turn lanes on uncontrolled approaches to intersections and driveways on urban two-lane and four-lane roadways. The guidelines reflect the circumstances for which the costs of right-turn lanes are justified by the operational and accident cost savings they provide to road users. The operational cost savings were those associated with the reductions

TABLE 56
RIGHT-TURN GUIDELINES FOR URBAN TWO-LANE ROADWAYS (*219*)

Roadway DDHV (vph)	Minimum Right Turn DHV (vph)															
	Within Existing ROW Roadway Speed (km/h)				ROW Cost = \$0.093/m^2 Roadway Speed (km/h)				ROW Cost = \$0.465/m^2 Roadway Speed (km/h)				ROW Cost = \$0.93/m^2 Roadway Speed (km/h)			
	40	56	72	89	40	56	72	89	40	56	72	89	40	56	72	89
100			65	30			70	40								
125	65	60	40	25	70	65	50	25			75	45				
150	60	50	35	20	65	55	40	20	75	75	60	35	95	95	90	50
200	50	45	30	15	55	45	30	15	65	65	40	25	80	80	60	30
400	40	35	20	10	40	35	20	10	40	40	30	20	55	55	40	20
600	35	30	15	10	35	30	15	10	35	35	25	15	45	45	35	15
800	30	25	15	10	30	25	15	10	30	30	20	10	35	35	30	15
1,000	25	20	15	10	30	25	15	10	30	30	20	10	35	35	30	15
1,200	25	20	15	10	30	25	15	10	30	30	20	10	35	35	30	15

Notes: DHV = design hourly volume; DDHV = directional design hourly volume; ROW = right-of-way; 1 km/h = 0.6 mph; 1 m = 3.3 ft.

TABLE 57
RIGHT-TURN GUIDELINES FOR URBAN FOUR-LANE ROADWAYS (*219*)

Roadway DDHV (vph)	Minimum Right Turn DHV (vph)															
	Within Existing ROW Roadway Speed (km/h)				ROW Cost = \$0.093/m^2 Roadway Speed (km/h)				ROW Cost = \$0.465/m^2 Roadway Speed (km/h)				ROW Cost = \$0.93/m^2 Roadway Speed (km/h)			
	40	56	72	89	40	56	72	89	40	56	72	89	40	56	72	89
100				35				60								
150	80	65	40	25	85	70	45	25			70	40				60
200	70	55	35	20	75	60	35	20	85	75	50	30	110	10	70	40
500	45	40	25	15	50	45	25	15	60	50	35	25	70	60	40	30
1,000	35	30	20	10	35	30	20	10	40	40	25	15	45	45	35	20
1,500	30	25	15	5	30	25	15	5	35	35	20	10	40	40	30	15
2,000	25	20	15	5	25	20	15	5	30	30	20	10	35	35	25	15
2,500	20	20	15	5	20	20	15	5	25	25	20	10	30	30	20	15
3,000	20	20	15	5	20	20	15	5	25	25	20	10	25	25	20	15

Notes: DVH = design hourly volume; DDHV = directional design hourly volume; ROW = right-of-way; 1 km/h = 0.6 mph; 1 m = 3.3 ft

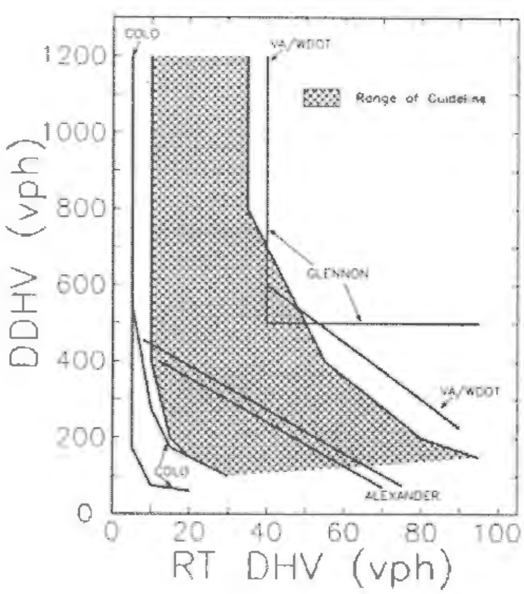


FIGURE 35 Comparison of right-turn lane guidelines for urban two-lane roadways (*219*).

in stops, delays, and fuel consumption experienced by through traffic. The accident cost savings were those associated with the reduction in accidents expected from the lower speed differentials between right turning and through traffic. Tables 56 and 57 list the guidelines for two-lane and four-lane facilities, respectively. Comparisons with guidelines developed by others (see Figures 35 and 36) indicated that the McCoy et al. guidelines are within the range of existing guidelines. The authors note that their guidelines are more definitive because they account for the effects of roadway speed and right-of-way costs.

McCoy and Bonneson (*220*) also developed volume warrants for free right-turn (FRT) lanes at unsignalized intersections on *rural* two-lane highways. The warrant was based on a cost–benefit analysis that determined the right-turn volumes required to justify the construction and maintenance of FRT. FRT lanes were not found to affect the frequency, severity, or types of accidents that occur at unsignalized intersections on rural two-lane highways. Thus, the benefits of FRT lanes are limited to improving the efficiency of right-turn movements. The results of the research indicate that design-year right-turn annual average daily traffic ranging from 440 to 825 vpd, depending on the percentage of trucks, is required to warrant an FRT lane (Figure 37). A design speed of 40 mph (64 km/h) was found to be the most cost-effective design speed for FRT lanes. However, design speeds up to 55 mph (89 km/h) do

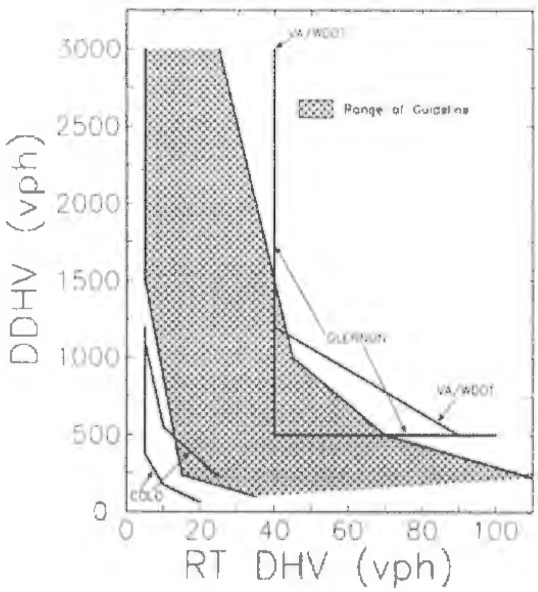


FIGURE 36 Comparison of right-turn lane guidelines for urban four-lane roadways (*219*).

not significantly reduce the cost effectiveness of an FRT lane.

Hasan and Stokes (*221*) developed guidelines for right-turn treatments at unsignalized intersections and driveways on rural highways in Kansas. Similar to McCoy and Bonneson (*220*), Hasan and Stokes considered operational and accident cost savings along with construction costs. They developed guidelines for full-width right-turn lanes and tapers. Tables 58 and 59 provide the guidelines for rural two-lane and four-lane highways, respectively.

TRUCKS

Mason et al. (*222*) discuss the geometric and operational considerations of large trucks at intersections. The key findings are

- Physical characteristics—Although the 1990 *Green Book* (*1*) currently includes 15 design vehicles, future truck combinations will probably follow some form of the Turner-type truck. Information provided regarding Canadian trucks indicates that the Canadian tractor–semitrailer combinations are similar to the AASHTO WB-62 and WB-67 types, and Canadian doubles are generally large than the AASHTO 1990 *Green Book* WB-60 double-trailer combinations.
- Offtracking—Turning roadway design speed governs the type and amount of offtracking that the truck–trailer units will generate. Fundamentally, low-speed offtracking decreases with increased turn radius, increases with increased turn angle, and increases with

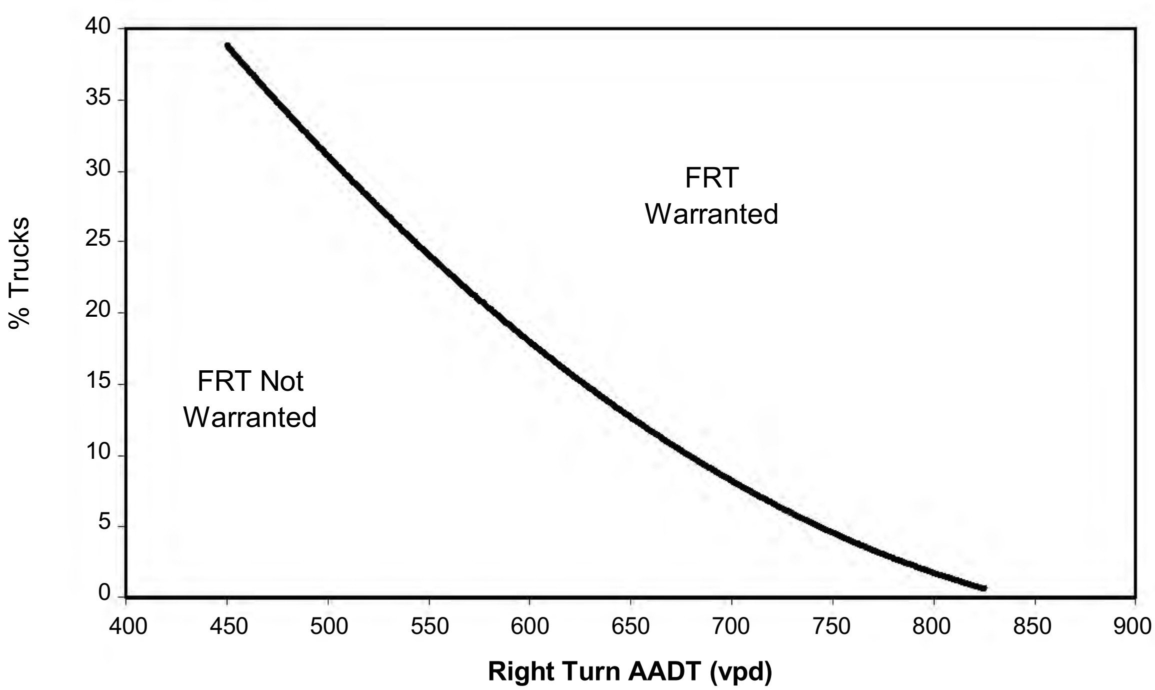


FIGURE 37 Volume warrant for free right-turn (FRT) lanes (*220*).

TABLE 58
RIGHT-TURN TREATMENT GUIDELINES FOR RURAL TWO-LANE HIGHWAYS (*221*)

Roadway DDHV (vph)	Minimum Right-Turn DHV (vph)[a]											
	Lane Roadway Speed (km/h)						Taper Roadway Speed (km/h)					
	64	72	81	89	97	105	64	72	81	89	97	105
200			73	35	20	15		83	30	14	8	7
300		120	41	24	15	12	85	40	19	9	7	6
400	200	52	30	19	12	11	27	27	14	8	6	5
600	50	26	20	14	10	9	12	13	9	6	5	4
800	25	16	15	11	9	8	8	8	7	5	4	3
1,000	14	12	11	9	8	7	6	5	5	4	3	3
1,200	10	9	9	8	7	7	4	4	4	4	3	3

Notes: DHV = design hourly volume; DDHV = directional design hourly volume; 1 km/h = 0.6 mph; 1 m = 3.3 ft.
[a]Minimum right-turn design hour volumes (vph) required to warrant right-turn treatments based on an assumed turning speed of 24 km/h.

TABLE 59
RIGHT-TURN TREATMENT GUIDELINES FOR RURAL FOUR-LANE HIGHWAYS (*221*)

Roadway DDHV (vph)	Minimum Right-Turn DHV (vph)[a]											
	Lane Roadway Speed (km/h)						Taper Roadway Speed (km/h)					
	64	72	81	89	97	105	64	72	81	89	97	105
300				75	19	19			55	25	9	9
400		145	75	40	16	15		65	30	17	8	8
500		95	57	32	14	13	140	50	25	14	7	7
600	160	65	42	26	12	12	80	30	18	11	6	6
800	70	37	28	19	11	11	40	18	12	8	5	5
1,200	25	20	18	14	8	8	14	10	8	6	4	4
1,600	15	14	13	10	7	7	8	6	6	5	3	3
2,000	10	9	9	8	6	6	6	6	4	4	3	3

Notes: DHV = design hourly volume; DDHV = directional design hourly volume; 1 km/h = 0.6 mph; 1 m = 3.3 ft.
[a]Minimum right-turn design hour volumes (vph) required to warrant right-turn treatments based on an assumed turning speed of 24 km/h.

TABLE 60
SUGGESTED WARNING SIGN PLACEMENT TO ACCOMMODATE DESIGN VEHICLES LARGER THAN AASHTO WB-50 TRUCK AT URBAN INTERSECTIONS (*222*).

Posted or 85th percentile speed (mph)	Distance from warning sign to potential hazard (ft)[a]						
	Condition A[b] (high judgment needed)	Condition B[c] (stop required)	Condition C[d] [deceleration to stated advisory speed (mph)]				
			10	20	30	40	50
20	250	—[e]	—[e]	NA	NA	NA	NA
25	325	—[e]	125	—[e]	NA	NA	NA
30	425	175	225	150	NA	NA	NA
35	500	250	325	250	100	NA	NA
40	600	325	450	375	225	NA	NA
45	675	425	600	500	325	175	NA
50	775	525	750	650	525	325	NA
55	850	650	900	825	675	500	225
60	950	775	1,075	1,000	875	675	425
65	1,025	900	1,225	1,200	1,050	800	600

Notes: NA = not applicable; 1 mph = 1.61 km/h; 1 ft = 0.305 m.
[a]All distances are based on the assumption that the warning sign is legible to drivers for 125 ft (38 m) in advance of the sign. For large [48-in. by 48-in. (122-cm by 122-cm)] signs, the legibility distance can be increased by 200 ft (61 m) and each of the entries in this table can therefore be reduced by 75 ft (23 m).
[b]Includes 2.0-sec Perception–Intellection–Emotion–Volition (PIEV) time.
[c]Includes 2.5-sec PIEV time and deceleration rates for driver with 70 percent breaking control efficiency.
[d]Based on comfortable deceleration rate equal to two-thirds of the deceleration rate used for Condition B.
[e]No suggested minimum distance provided; at these speeds, sign location depends on physical conditions at site.

trailer length. Double (twin) trailers 29 ft (8.8 m) long typically offtrack less than single-trailer combination trucks. Four methods for determining offtracking are offtracking plots and templates, computer models, offtracking charts, and offtracking equations.

- Channelization—Specific guidelines are not available to fully evaluate the need and scope of intersection channelization to accommodate trucks on turning roadways. It seems desirable to fit the pavement edges of the turning roadway to a spiral or taper geometry to minimize excess pavement and to conform more closely to a truck's offtracking.
- Sign placement—Table 60 provides advance placement criteria for warning signs. The modifications were based on increased stopping distances and comfortable deceleration rates for trucks. However, data are not available on whether trucks encounter any safety problems at signs placed in accordance with

existing criteria or whether there would be any safety benefits from adopting the modified criteria. The recommended advance warning sign distances for trucks could be reduced if antilock brakes were to become widely used.

Hummer et al. (*223*) preformed a limited study on the operations of large trucks. Computer simulation and manual observations at six intersections in California and New Jersey were used to investigate turns by large trucks at urban intersections. The authors noted that the simulation was limited because the differences among individual truck drivers, the reaction of the drivers of other vehicles in the traffic stream, and the speed of the turn were not modeled. The field observations were limited because they were based partially on a control truck with a professional driver knowledgeable of the purpose of the observations and because of the small samples of traffic-stream truck data gathered at some sites. The results showed that small curb radii, narrow lane widths, and narrow total street widths were among the geometric features associated with increased operational problems.

Fambro et al. (*224*) developed guidelines for intersection channelization to accommodate large truck combinations that represent the longer and wider trucks permitted by the Surface Transportation Assistance Act of 1982. The characteristics of these trucks were used in the California Truck Offtracking Model. The following design controls were determined: minimum turning radii, turning templates, cross street width occupied, turning roadway width, and channelization guidelines. The paper included tables that show the cross street width occupied by turning vehicles, the swept path width of the trucks, and minimum designs and channelization guidelines for turning roadways in a format similar to Table IX-4 of the *Green Book* (see Table 61).

USERS

Dewar (*225*) and Harkey (*226*) reviewed the characteristics of older pedestrians in relationship to intersections. They both commented that the walking speed of most older pedestrians is less than the assumed walking speed of 4 ft/sec (1.3 m/sec). Dewar cited a study that found the 85th percentile comfortable speed of 2.2 ft/sec (0.7 m/sec). Dewar suggested the following countermeasures to lower pedestrian accidents: increase the use of one-way streets to reduce the complexity of crossings for pedestrians and increase street lighting. In spite of the reduced pedestrian traffic at night, about one-third of the accidents occur in darkness. Harkey looked at left-turn, right-turn, and crossing maneuvers and provided suggestions on possible design changes that consider older driver and pedestrian characteristics.

TABLE 61
MINIMUM DESIGNS AND CHANNELIZATION GUIDELINES FOR TURNING ROADWAYS (*224*).

Angle of Turn (degrees)	Design Vehicle	Curb Radius (ft)	Width Turning Lane (ft)	Approximate Island Size (ft^2)
60	WB-50	200	27	250
	WB-55	200	22	160
	WB-70	200	22	160
	WB-100	200	27	160
	WB-150	—	—	—
75	WB-50	150	28	320
	WB-55	150	30	160
	WB-70	150	23	200
	WB-100	200	34	300
	WB-150	—	—	—
90	WB-50	150	30	670
	WB-55	200	38	900
	WB-70	150	22	560
	WB-100	200	40	900
	WB-150	200	54	260
105	WB-50	150	32	980
	WB-55	150	41	740
	WB-70	150	31	1,320
	WB-100	200	41	1,940
	WB-150	200	57	940
120	WB-50	150	40	1,640
	WB-55	200	45	3,400
	WB-70	150	39	1,600
	WB-100	200	48	2,580
	WB-150	200	60	1,740

Note: 1 ft = 0.305 m.

For left-turn problems, Harkey suggested the use of the following:

- Full positive-offset left-turn lanes [benefits include increased sight distance for left-turning drivers, improved channelization, and inclusion of a pedestrian refuge island. However, the solution requires intersections where there is an existing median of 20 ft (6.1 m) or more in width or where additional right-of-way can be purchased];
- Angle-entry bays (benefits include better orientation of vehicles in the turn bay); and
- Roundabouts (although the benefits of reduced speeds by all vehicles may be outweighed by the disadvantages of greater task loading and the challenges for older pedestrians because traffic never stops).

For right-turn maneuvers, Harkey suggests increasing the curb radius to improve maneuverability. He notes that if the curb-to-curb distance for pedestrians is dramatically increased, than a refuge island should be considered. Installing a raised channelized island is another potential countermeasure for problems associated with right-turn maneuvers.

Another area where older drivers have difficulty is with the crossing maneuver. Solutions include ensuring adequate sight distances on all legs, avoiding construction of intersections on horizontal and vertical curves, and providing an adequate length beyond the intersection for a lane that is being dropped along with adequate signing and marking well in advance to provide older drivers with sufficient decision and reaction time.

Harkey lists four major points that should always be considered when designing intersections for older road-users:

- Keep it simple—Although it is understood that the need for vehicle capacity has in the past, and will most likely continue in the future, dictate the number and type of lanes at an intersection, it is important to limit the number of lanes to reduce the decisions that have to be made by older drivers and to minimize the distance to be crossed by older pedestrians.
- Be consistent—It is important to be consistent in the design of intersections to avoid putting older drivers in situations that they would not normally expect. Where unique designs are incorporated, appropriate instruction must be provided in advance through traffic control devices.
- Develop alternatives as a system—No intersection operates solely as a function of geometrics, but rather as a system that incorporates both geometrics and traffic control devices. The proposed alternative designs must recognize this and address the traffic control that will be required.
- Do not create problems—In developing alternative designs for a specific problem (e.g., increased curb radii for older right-turn drivers), it is important not to create another problem (e.g., longer crossing distances for older pedestrians).

Material in "Building a True Community" (*35*) can provide insight into the needs of people with disabilities. For example, it recommends that 3.5 ft/sec (1.1 m/sec) be used as the assumed walking speed to better accommodate disabled pedestrians.

OLDER DRIVERS

Garber and Srinivasan (*227,228*) identified intersection design and operation parameters that significantly affect the accident involvement of the older driver. Statistical models were developed relating the risk of accident involvement to the traffic and geometric characteristics of the intersection at locations in Virginia. Police accident reports for intersection accidents involving drivers 50 years of age and older were obtained for four cities. Traffic and geometric data for the intersections were obtained from a survey of city engineers. The authors found that older drivers have a higher potential for committing a traffic violation during a turning maneuver, particularly when making left turns, when the predominant traffic violation of the older driver is failure to yield right-of-way, and that the involvement ratio for older drivers at intersections outside of cities is higher than for intersections within cities. The following conclusions were made from the results of their statistical modeling:

- An increase in the percentage of left-turn volume at an intersection increases the involvement ratio for the older driver. This effect is, however, reduced by the provision of a protected phase with left-turn lanes for a large percentage of left-turn volumes.
- An increase in amber time (caution light) for a given speed limit reduces the involvement ratio for the older driver. This implies that an amber period of 3 to 5 sec is probably not sufficient for the older driver, because of their longer reaction times.
- The involvement ratio of the older driver is much more dependent on the average annual daily traffic (AADT) than on the peak-hour volumes, most likely because older drivers travel less during peak hours than other age groups.

An FHWA report on older drivers (*7*) presents highway design information that can help accommodate the needs and capability of older road users. The report states that "these recommendations do not constitute a new standard of required practice. When and where to apply each recommendation remains at your discretion as the expert practitioner. The recommendations provide guidance that is firmly grounded in an understanding of older drivers' needs and capabilities, and can significantly enhance the safety and ease of use of the highway system for older drivers in particular, and for the driving population as a whole." The handbook includes recommendations on at-grade intersections, grade-separated interchanges, roadway curvature and passing zones, and construction/work zones.

Naylor and Graham (*229*) conducted a field experiment to determine an appropriate value for PRT for a driver turning left from a Stop-controlled approach. Subjects were videotaped as they entered two rural and two urban Stop-controlled intersections. The 85th percentile decision–reaction time for the older group (average age of 69.3 years) was 1.86 sec and for the younger group (less than 30 years old) was 1.66 sec. Both times were less than the 1990 *Green Book* (*1*) design value of 2.0 sec.

ACCIDENTS/CONFLICTS

Farber (*230*) developed a model that estimated the relative hazard to passenger cars stopping to turn left at an intersection hidden by a vertical curve. Because of the limited sight distance, following cars might not be able to see the left-turning vehicle in time to stop on wet pavement. The results indicated that the conflict rates increased rapidly with decreasing sight distance. Farber estimated between 0.32 and 101 conflicts per year could be expected depending on sight distance and daily volume. Countermeasures proposed for these types of conflicts include reducing speed, appropriate signing, and increases in pavement friction. The author concluded that these countermeasures are at least as effective in reducing conflicts at sites with limited sight distance as improving sight distance.

Driveways close to intersections [within 98.4 ft (30 m)] result in inappropriate left turns that create safety and operational concerns. Parsonson (*231*) reports that interviews with state DOT traffic engineers in the southeast suggest that easy-to-implement countermeasures are seen as either hazardous in themselves or else ineffective. He recommends a prefabricated raised median treatment of 3.5 in. (90 mm) high and 1 ft (305 mm) wide that could be used without widening the road or narrowing the lanes. The design needs to be field tested under controlled conditions for effectiveness and safety.

The combination of high-speed operation and only partial access control can have an adverse impact on the safety of rural expressways. Bonneson et al. (*232*) identified the measures used by state highway departments to mitigate these impacts. One such measure is the access control policy, which is used to regulate the frequency and location of all access to the expressway. Measures used with at-grade intersections along the expressway include traffic control devices and geometric design features (Table 62). The authors state that one of the more novel corrective measures is the offset left-turn bay, which minimizes the sight distance blockage created by opposing left-turn vehicles.

TABLE 62
SUMMARY OF CORRECTIVE MEASURES AT RURAL UNSIGNALIZED INTERSECTIONS (*232*)

Corrective Measures	No. of States	Percent of States[a]
Signalization	17	74
Traffic signal control	12	42
Flashing beacon	11	48
Intersection control beacon	9	39
Stop sign beacon	2	9
Hazard identification beacon	2	9
Signing Improvements	8	35
Advance signing	6	26
Increase sign size	2	9
Reduce sign clutter	1	4
Exclusive Lanes for Turning Traffic[b]	7	30
Grade Separation/Interchange	5	22
Reduce Speed Limit	4	17
Partial Lighting	3	13
Rumble Strips	2	9

[a]Frequencies based on responses from 23 states.
[b]Treatments mentioned include: add right-turn bay, lengthen left-turn bay, add median acceleration lane, offset left-turn lanes, and prohibit turns by closing median.

Weerasuriya and Pietrzyk (*233*) developed conflict tables for unsignalized three-legged intersections based on a study of traffic conflicts and crash history at 38 intersections in west-central Florida. The values in Table 63 can be used to estimate the relative safety effectiveness of unsignalized three-legged intersections. Should an intersection exhibit higher conflict rates than those listed in the tables, the data can be used in the development, justification, implementation, and evaluation of highway safety improvement projects. The five-step procedure that can be used to generate an estimate of the yearly number of crashes at the intersection attributable to each of the conflict types is listed as follows:

1. Identify intersection grouping according to the number of legs, signalization (signalized or unsignalized), and lanes (2×2, 2×4, etc.).
2. Observe conflict counts for the subject intersection following the procedures described in Parker and Zegeer (*266*). Add conflict counts for each conflict type during an observation period of 2 h per intersection leg to find the total conflict count for each conflict type. This total conflict count would equate to a 4-h conflict count observation for each intersection.

TABLE 63
FLORIDA-BASED CONFLICTS (*233*)

Conflict Type	Mean	Variance	C_{90}[a]	C_{95}[a]	A[b]	B[b]	C[b]
Signalized, 3-Legged, 2 × 2 Intersections							
1. Left-turn, same direction	4.67	76.46	13.84	21.70	**	**	**
2. Slow vehicle, same direction	10.75	195.30	28.05	38.88	0.01360	0.23727	0.00103
3. Lane change, same direction	0.00	0.00	—	—	**	**	**
4. Right-turn, same direction	3.92	58.45	11.71	18.68	**	**	**
5. Left-turn, opposing direction	0.75	6.75	1.82	4.37	**	**	**
6. Left-turn-from-left, cross traffic	1.08	3.36	3.13	4.72	0.00689	0.00207	0.00057
7. Through, cross traffic from left	0.17	0.15	0.50	0.88	**	**	**
8. Right-turn, cross traffic from left	0.00	0.00	—	—	**	**	**
9. Left-turn, cross traffic from right	0.50	3.00	1.22	2.91	0.00501	0.00090	0.00028
10. Through, cross traffic from right	0.17	0.15	0.50	0.88	**	**	**
11. Right-turn, cross traffic from right	0.08	0.08	0.20	0.49	**	**	**
12. Conflicts other than 1 through 11	0.58	4.08	1.42	3.40	0.00394	0.00082	0.00019
1 through 4, same direction	19.33	786.97	52.86	75.64	0.00001	**	**
7 plus 10, through, cross traffic	0.33	0.24	0.92	1.32	**	**	**
Unsignalized, 3-Legged, 2 × 4 Intersections							
1. Left-turn, same direction	1.92	10.45	5.53	8.32	0.03306	0.11418	0.00984
2. Slow vehicle, same direction	13.25	111.11	27.26	33.92	0.07530	4.59659	0.03570
3. Lane change, same direction	1.50	2.27	3.46	4.51	0.03444	0.02375	0.00926
4. Right-turn, same direction	2.83	8.52	6.62	8.67	0.02755	0.08402	0.00911
5. Left-turn, opposing direction	0.25	0.20	0.74	1.14	0.04132	0.00314	0.01366
6. Left-turn-from-left, cross traffic	0.83	2.33	2.46	3.82	**	**	**
7. Through, cross traffic from left	0.08	0.08	0.20	0.49	**	**	**
8. Right-turn, cross traffic from left	0.00	0.00	—	—	**	**	**
9. Left-turn, cross traffic from right	0.33	0.24	0.92	1.32	0.03673	0.00327	0.01214
10. Through, cross traffic from right	0.08	0.08	0.20	0.49	0.03673	0.00112	0.01214
11. Right-turn, cross traffic from right	0.50	0.64	1.42	2.09	**	**	**
12. Conflicts other than 1 through 11	6.08	307.17	17.25	34.70	**	**	**
1 through 4, same direction	19.50	204.09	38.57	47.31	0.00014	0.00002	**
7 plus 10, through, cross traffic	0.17	0.15	0.50	0.88	0.00006	**	**
Unsignalized, 3-Legged, 2 × 6 Intersections							
1. Left-turn, same direction	11.43	344.11	32.58	48.35	**	**	**
2. Slow vehicle, same direction	35.43	575.96	67.54	81.80	0.04287	5.62256	0.00792
3. Lane change, same direction	6.50	43.04	15.04	19.60	0.03062	0.14237	0.00237
4. Right-turn, same direction	16.00	347.23	39.67	53.45	**	**	**
5. Left-turn, opposing direction	3.14	15.36	8.05	11.03	0.03936	0.15990	0.00886
6. Left-turn-from-left, cross traffic	1.57	5.80	4.39	6.39	0.02543	0.05253	0.00841
7. Through, cross traffic from left	0.29	0.68	0.81	1.63	**	**	**
8. Right-turn, cross traffic from left	0.00	0.00	—	—	**	**	**
9. Left-turn, cross traffic from right	0.64	0.86	1.76	2.51	0.10018	0.04732	0.04482
10. Through, cross traffic from right	0.36	0.55	1.08	1.77	**	**	**
11. Right-turn, cross traffic from right	2.64	8.71	6.42	8.57	0.03306	0.08912	0.00914
12. Conflicts other than 1 through 11	3.86	116.29	11.11	21.82	0.00907	0.08803	0.00067
1 through 4, same direction	69.36	1734.55	125.18	148.9	0.00001	**	**
7 plus 10, through, cross traffic	0.64	1.02	1.81	12.66	0.00007	**	**

Notes: Conflict counts were obtained during a 4-h observation period on a weekday (Monday through Thursday, excluding holidays) between 7 AM and 6 PM under dry pavement conditions. Counts do not include secondary conflicts.
Blanks (—) indicate that these conflict types are so rare that any number observed at an intersection should be considered abnormal.
Asterisks (**) indicate that either zero crashes were reported during 1992–1994 period or that the constant is very small.
[a]Abnormally high conflict counts.
[b]Crash/conflict constants.

3. The estimate of the yearly (days including only Monday through Thursday, nonholiday, and during daylight hours—7:00 AM to 6:00 PM) number of crashes for a particular conflict type is

$$Y_o = \text{Conflict Count} \times A$$

where A equals 201 × 5.5 × (mean of crash/conflict ratio).

The value 201 is the number of nonholiday weekdays (Monday through Thursday) in a typical 365-day year [i.e., (365 × 4/7) − 7 = 201]; the value 5.5 is to extrapolate a 2-h count to an 11-h day count. The value of A, which is a constant for a conflict type of a given intersection, can be obtained from Tables 2–4.

4. The variance associated with this estimate is from ref. *233*.

TABLE 64
TOTAL ACCIDENTS BY INTERSECTION TYPE IN RURAL MUNICIPALITIES (*234*)

Intersection Type	Total No.	Average Accident Rate[a]
Four-way	1,517	1.35
T-Type	373	0.80
Y-Type	127	1.22
Offset	54	0.58

Notes: Total includes both Stop and signalized intersections. Total average accident rate for study = 1.13.
[a]Accidents per million entering vehicles.

TABLE 65
ACCIDENT RATES BY INTERSECTION TYPE IN URBAN LOCATIONS (*234*)

	Average Accident Rate[a]	
Average daily traffic	T-Type	Four-Way
<5,000	1.3	1.3
5,000 to 10,000	1.6	1.9
10,000 to 20,000	2.7	3.0
>20,000	4.2	8.0

Note: Includes only intersections with Stop signs; data were not available for signalized intersections.
[a] Accidents per million entering vehicles.

$$\text{var}\,(Y_o) = B + [(\text{conflict count})^2 \times C]$$

where B = var(conflict) × var(crash/conflict ratio) + (mean of crash/conflict ratio)^2 × var(conflict) and C = var(crash/conflict ratio). The values B and C, which are constants for a conflict type of a given intersection, can be obtained from Tables 72, 73, and 74 from ref. *233*.

5. A 95th percentile reliable estimate for the number of crashes caused by the specific conflict type is found using

$$(Y_0) - 2\sqrt{\text{var}\,(Y_0)}$$

Kuciemba and Cirillo (*234*) produced a synthesis of findings on the relationship between accidents and highway geometry for intersections. Table 64 cites the total number of accidents by intersection type in rural municipalities, and Table 65 cites accident rates for urban locations (including both Stop- and signal-controlled intersections). For a comparative basis, the average accident rate for all intersection accidents in the study was 1.13 accidents per million entering vehicles. A study at urban intersections with Stop signs found accident rates very similar for four-way and T-type design, with an ADT of under 20,000. Above 20,000, the accident rate doubled for four-way, when compared with T-type intersections (see Table 64). The authors also included a table that provided expected accident reduction for sight distance improvements and tables on the minimum number of passing accidents required to justify design treatments.

A recent NCHRP project produced an *Accident Mitigation Guide for Congested Rural Two-Lane Highways* (*37*), which includes a review of the effectiveness of several countermeasures used to improve safety at rural intersections.

Recent research projects have developed crash models for two-lane rural segments and intersections (*235*) and four-lane by two-lane Stop-controlled intersections (*236*) and two-lane by two-lane signalized intersections (*236*).

CHAPTER SIX

INTERCHANGES

INTERCHANGE DESIGN

Leisch (*237*) presented essential criteria for planning and designing a new freeway facility or considering operational and design improvements to an existing facility. He notes that while the operational and design criteria discussed in his paper are present in various chapters of the 1990 *Green Book*, his intention is to clarify their application in freeway and interchange planning and design. The following list summarizes his criteria:

- System Criteria
  - Basic number of lanes—The constant number of lanes assigned to a route, exclusive of auxiliary lanes.
  - Lane balance and auxiliary lanes—Occurs at exits when the number of lanes approaching is equal to one lane less than the combined number departing. At entrances, the combined number of lanes after the merge should either be equal to or one lane less than the total number of lanes approaching the merge.
  - Route continuity—The provision of a directional path along and throughout the length of a designated route.
- Interchange Considerations
  - Appropriate interchange form—Consideration of the appropriate form may include classification of intersecting facilities, volume and pattern of existing and future traffic, physical constraints and right-of-way considerations, environmental requirements, local access and circulation considerations, construction and maintenance costs, and road-user costs.
  - No weaving within interchange—Weaving within an interchange exhibits high accident experience and poor operational characteristics that usually affect not only entering and exiting traffic but mainline flow as well.
- Operation Uniformity Criteria
  - Right exits and entrances—Satisfies driver expectancy and keeps slow-moving vehicles from left lanes and avoids weaving across all lanes of the freeway.
  - Single exit per interchange in advance of crossroad—Simplifies the driver's task by providing only one decision point on the freeway and giving the driver a view of the exit ramp well in advance.
  - Simplified signing—Can exist when exits are in advance of the crossroad and are on the right.
- Ancillary Guidelines
  - Decision sight distance—The distance at which a driver can perceive a decision point along the freeway.
  - Freeway and ramp speed relationship—Refers to the distance required for the driver to decelerate the vehicle from the speed of the freeway to the speed of the controlling curve of the ramp.
  - Ramp sequencing or spacing requirements—Provided in the *Green Book* and based on design requirements and capacity relationships.

Garber and Fontaine (*238*) developed general guidelines to aid designers in the selection of the optimum interchange type at a given location. The general guidelines (cited here) provide a starting point for analysis.

- Right-of-Way Availability—The interchange type selection guidelines based on right-of-way issues were developed primarily from the literature review. The survey results helped to further validate the information found in the literature review and also influenced the formulation of guidelines. Based on these two sources, the following guidelines were developed:
  - When the available right-of-way is limited, single-point urban interchanges (SPUIs) or diamonds are most appropriate, because they can be built in a limited right-of-way.
  - In situations where the right-of-way is restricted in one or more quadrants, the partial cloverleaf should be considered.
  - Full cloverleafs require an extensive amount of right-of-way, due to the presence of the loop ramps. The amount of land required for the full cloverleaf increases significantly as the design speed for the loop ramps increases. Thus, full cloverleafs may not be suitable for the application in urban areas or other situations where the amount of right-of-way available is limited.
  - Directional interchanges require the largest amount of right-of-way and are usually only justified for freeway-to-freeway connections.

- Construction Cost—The construction cost guidelines were developed primarily from the literature review. The survey results helped to further validate the information found in the literature review and also influenced the formulation of the guidelines. Based on these two sources, the following guidelines were developed:

- Cost figures for interchanges are very site-specific. Topography, land-use, and environmental concerns can make identical interchange designs have very different final costs depending on the site.
- Generally speaking, the diamond has the lowest cost of the interchange types, due to its small structure and the limited amount of right-of-way required.
- The cost of SPUI is generally 10 to 20 percent higher than for a diamond, due to the large structure that must be constructed. This can result in a very large bridge span (mainline over crossroad) or a butterfly-shaped structure (mainline under crossroad), which can cost considerably more than a conventional diamond interchange. Although construction costs for the SPUI structure are somewhat greater than for diamond interchanges (DIs), the higher cost is mitigated somewhat by the reduced right-of-way costs for the SPUI, especially in urban areas.
- Directional interchanges have the highest construction cost of all interchange types, due to the large structures involved and the extensive right-of-way they require. They are generally justified only when high speeds and large capacities are needed.

• Traffic and Operational Issues—Guidelines for interchange type selection based on operational issues were developed based on the literature review and operational analysis. The guidelines based on the literature review are

- When the arterial coordination is a major priority, the SPUI should be considered. The SPUI is easier to coordinate with other signals on an arterial route than a diamond, because it requires that only one signal be coordinated, rather than two.
- Full cloverleafs without collector–distributor roads should be used only when weaving volumes are small and right-of-way is not a concern, such as in rural areas.

The guidelines developed based on the operational analysis are

- The diamond interchange should be used when traffic volumes are very low (under 1,500 vph at peak hour entering volume). In these cases, signals usually are not warranted, and delays are very low with an unsignalized system.
- In cases where volumes are between 1,500 and 5,500 vph, the SPUI should be used instead of the diamond. The diamond has consistently higher delays due to the two-intersection configuration of the interchange.
- The delay at the SPUI increases significantly when the ramp left turns are unbalanced. There are also some indications that unbalanced mainline left turns may increase delay at the SPUI. Thus, proposed designs should be carefully analyzed when either of these conditions is present.
- The partial cloverleaf provides greater capacity than the SPUI or the diamond when the peak entering volume is between 1,500 and 2,500 vph. The signalized delay at the partial cloverleaf is less than the SPUI and the diamond for all cases tested. All components of the partial cloverleaf performed at a higher level of service than the SPUI or diamond at 1,500 and 2,500 entering vph. Weaving operations are the critical component of high-volume, partial cloverleaf interchanges.
- Weaving operations are critical at full cloverleafs and when provided at partial cloverleafs. The level of service of the weaving areas begins to decline as the number of weaving vehicles approaches 1,000 vph. This indicates that full cloverleafs with collector–distributor roads, semi-directional interchanges, or directional interchanges should be used when weaving volumes approach 1,000 vph. It also shows that the partial cloverleafs should be designed without weaving areas when a condition like this occurs.
- In suburban areas, the volumes and traffic patterns can change dramatically in short periods of time. Delay at SPUIs and diamonds can change dramatically, depending on traffic distributions; therefore, signal timings must be optimized in these situations to minimize delays.

• Other Issues—The remaining guidelines were developed principally from the literature review. The accident analysis did play some role in the development of the first guideline.

- Loop ramps generally have a worse safety record than other ramp types and should generally be avoided where possible. Weaving areas have a poor safety record, especially when collector–distributor roads are not provided. Particular attention should be given to the design of weaving areas of cloverleaf interchanges, due to these safety concerns.
- When two roads intersect at a large skew angle, use of the SPUI is not recommended. The skew angle will result in high construction costs for the SPUI and also result in reduced sight distances at the interchange.
- Pedestrians are not easily accommodated by the SPUI without greatly increasing delay at the interchange. DIs can accommodate high pedestrian volumes much better.

- Full cloverleafs are the minimum facility that can be provided for two access-controlled facilities. However, the use of full cloverleafs for system interchanges is not recommended unless the weaving volumes are very low. Usually, directional interchanges provide better service for freeway-to-freeway connections.
- Trumpets should be used when three intersecting legs are present.
- When frontage roads are present, the diamond is preferred over the SPUI. A fourth phase would be required to handle the frontage roads at the SPUI, and this would significantly increase overall delay at the interchange.
- Interchange uniformity should also be considered when making interchange type selections. Interchange uniformity along a route can aid drivers in identifying where they need to enter or exit and can help reduce driver confusion.

The sources used to develop the guidelines included a literature search, a nationwide survey of state engineers, a review of 10 interchanges in Virginia (both delay and accident analyses), and a computer simulation of the interchange types. They also developed two flow diagrams that are intended to assist in selecting an initial interchange type; however, the flow charts do not include factors that could be very important in the selection of the optimum interchange type, such as topography, community impact, cost, and environmental concerns. Author recommendations for future research included

- Conducting a detailed cost analysis of the various interchange types to aid in interchange type selection.
- Identifying a common measure of effectiveness for different interchange types to allow for better comparisons.
- Completing a comprehensive accident analysis of the various interchange types to determine safety advantages and disadvantages related to interchange type.
- Conducting an operational study to determine the effect collector–distributor roads have on weaving operations.

Reviews of freeway designs were reported by Leisch (*239*) and Lamm et al. (*240*) in 1993. Leisch presented an historical perspective of U.S. freeway and interchange design, whereas Lamm et al. reviewed interchange planning considerations and interchange types for several countries including Australia, Austria, Germany, South Africa, Switzerland, Greece, Ireland, and Norway. Discussions on grade-separated intersections (sometimes called urban interchanges) are contained in chapter 5.

Single-Point Urban Interchange

Messer et al. (*241*) reported on current practices in design and traffic operations of existing SPUIs and developed

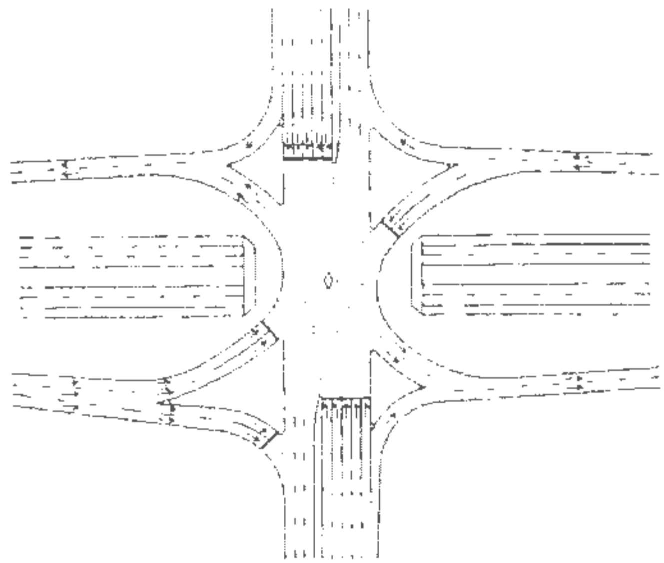


FIGURE 38 Single-point urban interchange (SPUI) (*241*).

guidelines for the design, traffic operational analysis, and cost-effectiveness of SPUIs in *NCHRP Report 345*. Their report provided information on historical development, typical geometric and bridge design, observed traffic operations, and general traffic engineering applications. The authors developed a list of 10 items believed to be the most critical in providing a good SPUI design, based on the results of the field surveys conducted at 36 SPUIs located in 13 states. The recommended principal SPUI design objectives are as follows:

- Size the interchange to provide adequate capacity to satisfy vehicular traffic demand expected for the design year in a safe and efficient manner.
- Select the most desirable grade separation type, overpass or underpass, for existing conditions. Simple, multispan bridges are preferred.
- Provide a bridge design that can efficiently add two future main lanes without major structural modification or significant impact on mainline traffic.
- Provide a design that can be readily expanded to a full 6-2-2-2 configuration during the design lift of the interchange (Figure 38).
- Provide a design that can be efficiently constructed, given site-specific conditions, in a minimum amount of time and with a minimum amount of traffic interruption.
- Provide a design sensitive to the local aesthetics and environment.
- Provide adequate visibility and sight distance of the critical geometric and operational features, both day and night.
- Provide facilities appropriate to serve the pedestrian traffic demand expected for the site. Minimize pedestrian impacts on traffic capacity.

- Provide the traffic control devices best suited to fulfill the needs of unfamiliar motorists operating on this class of interchange.
- Obtain adequate rights-of-way to satisfy design year traffic demand for the traffic movements at desirable operating speeds.

Merritt (*242*) used information from *NCHRP Report 345* (*241*) and other literature to produce a summary of visibility issues and geometric design features for SPUIs. He concludes that the complexity of the SPUI design requires careful selection of all design feature dimensions and an awareness of the impacts of design decisions on traffic operations and structural costs.

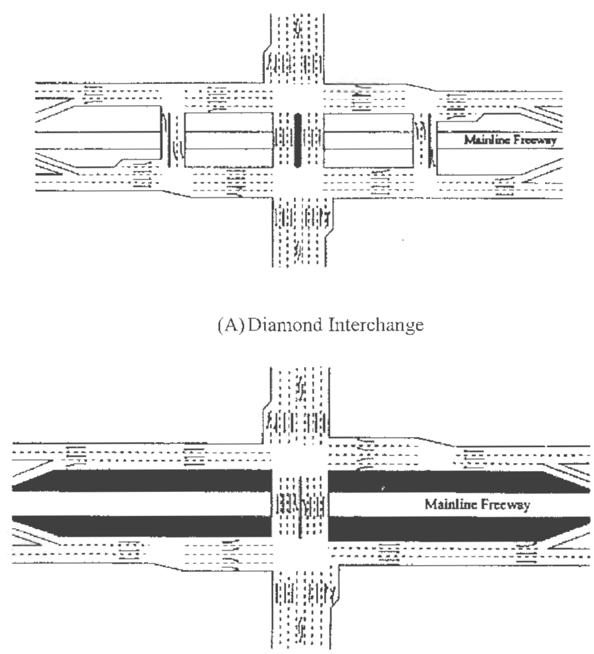
Dorothy et al. (*243*) also conducted a literature review and survey of SPUIs and included a field review of selected SPUI to evaluate their appropriateness for use in Michigan. They found the following:

- An SPUI with the crossroad going over the freeway was found to be the preferred design. The best placement of the traffic signal heads occurred in designs in which the crossroad went over the freeway, allowing the signal heads to be located on a single overhead tubular beam.
- When the freeway goes over the crossroad, sight distance is a concern.
- An SPUI without dedicated U-turn lanes appeared to accommodate U-turns as well as those with dedicated U-turn lanes. Thus, the smaller designs were observed to function better than the larger designs and right-of-way requirements are less with the smaller designs.
- Several engineers expressed strong opinions that the use of continuous frontage roads with an SPUI counteracts the advantages of the design.
- Because traffic is always moving through the intersection, pedestrians find it extremely difficult to cross.
- The need for pavement markings is paramount; however, markings can overlap and cause driver confusion.
- The use of channelized islands to help guide drivers through the intersection was determined not to be an effective solution in Michigan due to snow removal requirements.

Comparisons of Interchange Designs

Several papers compared the operational performance of different interchange forms. For example, Hook and Upchurch (*244*) found that SPUIs have significantly higher saturation flow rates for the exclusive left turn from the ramp, but not for the left turn from the arterial or the arterial through movement as compared with conventional DIs. There was no significant difference in start-up lost times between the two interchange forms. Single-point DIs have significantly higher clearance lost times for the ramp left turn and the arterial through movement. Fowler (*245*) examined the relative traffic-carrying capabilities of the tight-diamond interchange (TDI) [defined as having less than 250 ft (76.3 m) between ramp intersections] and the SPUI. He found that the relative performances of the TDI and the SPUI are highly dependent on the characteristics of the intersecting movements. In very tight configurations, the SPUI provides greater capacity for most traffic volume conditions than does the TDI. In addition, the SPUI's performance is less susceptible to differences in traffic patterns.

Dorothy et al. (*246*) used TRAF-NETSIM to operationally compare the DIs to the Michigan urban diamond interchange (MUDI) (Figure 39). Operationally, the MUDI was superior to the DI in most cases. The benefits were greater at higher saturation levels and high percentages of vehicles desiring to turn left onto the arterial. The authors also noted that the MUDI configuration does not transfer delay to downstream nodes, whereas the DI with frontage roads appears to affect the operation of these nodes.



(A) Diamond Interchange

(B) Michigan Urban Diamond Interchange (MUDI)

FIGURE 39 Typical diamond and MUDI interchanges (*246*).

Smith and Garber (*247*) evaluated and compared the safety and operational characteristics of the SPUI and the DI and developed guidelines (Table 66) that identify traffic and geometric conditions that favor one over the other. The guidelines were based on surveys of state traffic engineers,

TABLE 66
COMPARISON OF THE RESULTS FOR THE TUDI, SPUI, AND LHESS (*248*)

Characteristic	TUDI	SPUI	LHESS
Right-of-Way Requirement	Moderate	Moderate	Low
Costs	Moderate	High	Moderate
Sight Distance Requirements	Low	Moderate	High
Length of Vertical Curves	Low	Moderate	Low
Driver Expectancy	Meets	Violates Slightly	Violates
Accommodation of Pedestrians	Good	Poor	Good
Accommodation of Heavy Vehicles	Poor	Good	Poor
Operation Under Varying High-Volume Scenarios	Good	Fair	Poor

Notes: TUDI = tight urban diamond interchange; SPUI = single-point urban interchange; LHESS = left-hand exit single signal.

a literature review, and conversations with engineers experienced with the interchange types. The authors recommended that a more thorough examination of operational and safety characteristics of existing interchanges be done and that the cost-effectiveness of the SPUI and DI be compared to provide design engineers with more information for decision making.

Pate and Stover (*248*) investigated three interchange options for densely developed areas—tight urban diamond interchange (TUDI), SPUI, and the left-hand exit single signal. They concluded that, in general, TUDI interchange was found to be the best design alternative under urban conditions. It offers the greatest flexibility in operation and future expansion at a lower cost than the SPUI. The left-hand exit single signal design is not recommended because of its violation of driver expectancy. Their comparisons are listed here.

- Interchange Selection
  - When adjacent land use is such that restricted right-of-way is created, the SPUI should be seriously considered because, in general, it uses less land than the DI.
  - When it is necessary to provide a phasing system that takes into consideration pedestrian movement across the arterial, the DI is preferred, as the SPUI design does not accommodate this pedestrian traffic easily (see the following guideline section for specific design suggestions).
  - It is quite clear that when the interchange is to be constructed at locations with continuous frontage roads, the DI is preferred as the SPUI is less efficient at such locations, because the SPUI's through movement requires an additional fourth phase that significantly increases the overall interchange delay.
  - When there is a large skew angle between the intersecting roadway alignments, the DI is preferred; the SPUI is generally more expensive to construct because of the more extensive structure required. When skew angles exceed 30 degrees, AASHTO recommends taking extreme caution, as it not only significantly adds to the costs of the SPUI but also increases clearance distance and, therefore, lost time; it negatively affects sight distance as well.
  - At locations where a large percentage of heavy-truck traffic exists, it may be appropriate to use an SPUI, as the large turning radii allow for more efficient dual left turns of trucks side by side.
- Specific Design
  - In extreme cases when a SPUI must be designed to accommodate pedestrian traffic across the arterial, consider the following:
    - Provide a 3.1-m (10-ft) median (minimum size) on the arterial for pedestrian refuge, and design the phasing of the interchange to allow a pedestrian to cross halfway during the arterial left-turn phase and cross the remainder of the street during the following off-ramp phase or vice versa.
    - Provide an additional pedestrian phase, actuated by push buttons, where all traffic signal indications for vehicular traffic are red.
  - Whenever possible avoid signalizing the off-ramp right-turn vehicles at SPUIs. A Yield sign should be provided with an adequate acceleration lane. The yellow plus all-red clearance intervals will be greatly reduced when such a sign is used, thus improving the capacity of the interchange by increasing the green-to-cycle ratio.
  - In the SPUI design, give special consideration to the visibility between the off-ramps and the crossroad, as visibility of the oncoming traffic from the left is reduced at the SPUI, and drivers approaching the bridge from the off-ramp must rely on all traffic obeying the signal.
  - Pay distinct attention to signing and striping to reduce confusion and possible wrong-way maneuvers. A raised island in the middle of the signalized intersection provides positive delineation and reduces this confusion.

Design of Pedestrian Facilities at Interchanges

Zeidan et al. (*249*) developed guidelines for the design of pedestrian facilities at interchanges. The development of the guidelines was based on a literature review, survey of current practice, the observation of pedestrian behavior at interchanges, review comments from highway designers, and cost–benefit analysis of sidewalks on bridges. The guidelines address sidewalks, crossings, traffic control devices, and illumination as they pertain to the accommodation of pedestrians at interchanges. The authors made the following conclusions regarding the design of pedestrian facilities at interchanges:

- Pedestrian facilities should be considered in the planning of an interchange. The design of these facilities should provide for pedestrian convenience and safety.
- Pedestrian paths should be accessible, continuous, and direct; they should accommodate persons with disabilities; and should be adequate to accommodate the pedestrian volumes.
- Sidewalks may be provided on bridges based on a cost–benefit analysis and/or based on land-use and roadway functional classification. Where appropriate, pedestrians on such bridges should be protected by guardrails.
- Crosswalks should be provided at ramps to ensure continuity. These crosswalks, however, should be unmarked unless they are on a designated school route. Sidewalks to ramp crosswalks should be aligned to direct pedestrians along a minimum-exposure time path across the ramp.
- Cross-street crosswalks should not be provided within the interchange.

The report, *Design and Safety of Pedestrian Facilities* (*33*) also commented about pedestrians at expressway ramps, stating that the "hazard to pedestrians at ramp intersections is often difficult to correct . . . . however, the level of hazard at many of these intersections can be lessened through the use of appropriate traffic-control devices (e.g., warning signs) to reduce vehicle speeds and alert motorists and pedestrians. In some instances, pedestrian barriers, modified signal timing (e.g., longer vehicle clearance intervals), or even grade separation (e.g., pedestrian overpasses) in extreme situations may be needed to reduce a serious pedestrian safety problem."

The PROWAAC report (*35*) provides discussion on designs for people with disabilities. The report is available online at www.access-board.gov.

RAMPS

Harwood and Mason (*250*) explored issues inherent in the selection of appropriate ramp design speeds and geometrics to avoid interchange operational and safety problems. They noted that a 1990 FHWA study (*251*) concluded that the current AASHTO design policy provides an adequate margin of safety for both passenger cars and trucks on horizontal curves as long as the design assumptions on which the AASHTO policy is based are not violated. In particular, it is important that trucks not travel faster than the design speed on curves with relatively low design speeds. The FHWA study concluded that the current AASHTO horizontal curve design policy was adequate for both passenger cars and trucks traveling at or below the design speed of the highway. However, in some cases, a vehicle with very poor tires on a poor wet pavement could skid, or a vehicle with a worst-case rollover threshold could roll over, at only a few miles per hour above the design speed. Furthermore, the research found that skidding or rollover was most likely to be critical for curves with lower design speeds, such as ramps. Table 67 summarizes vehicle speeds at impending skid and impending rollover for several critical scenarios.

Harwood and Mason stated that design policy for horizontal curves should ensure an adequate margin of safety against both rollover and skidding at the travel speeds actually used by vehicles on a particular horizontal curve. In other words, it is not enough just to select a design speed for a ramp to fit the physical constraints of the site. There is also a need to determine an anticipated operating speed for the ramp. If a substantial percentage of vehicles are expected to travel faster than the design speed, there is a need to change to a higher design speed or to incorporate effective speed-control measures in the design. The guidelines listed here should be considered in selecting a potential design speed for a ramp.

TABLE 67
VEHICLE SPEED AT IMPENDING SKID AND ROLLOVER UNDER CRITICAL CONDITIONS (*250*)

Design Speed mph (km/m)	Maximum *e**	Passenger Car Speed mph (km/m) At Impending Skid (wet)	Passenger Car Speed mph (km/m) At Impending Rollover	Truck Speed mph (km/m) At Impending Skid (wet)	Truck Speed mph (km/m) At Impending Rollover
20 (32.2)	0.08	32.5 (52.3)	45.3 (72.9)	26.8 (43.1)	24.7 (39.8)
30 (48.3)	0.08	47.1 (75.8)	69.6 (112.1)	39.0 (62.8)	37.9 (61.0)
40 (64.4)	0.08	61.8 (99.5)	94.8 (152.6)	51.3 (82.6)	51.6 (83.1)
50 (80.5)	0.08	76.8 (123.6)	121.1 (195.0)	63.9 (102.9)	66.0 (106.3)
60 (96.6)	0.08	95.2 (153.3)	152.2 (245.0)	79.3 (127.7)	82.9 (133.5)
70 (112.7)	0.08	118.0 (190.0)	191.5 (308.3)	98.5 (158.6)	104.3 (167.9)

*Maximum superelevation rate (*e*).

Consider the following in selecting the design speed for an off-ramp:

- Consider physical and economic constraints in selecting a tentative design speed for the ramp. Use the upper or middle range, if possible. It is especially important to avoid the lower range of ramp design speeds on ramps that will carry substantial volumes of truck traffic.
- Identify the most critical curve on the ramp (usually, but not necessarily, the first curve downstream of the gore area).
- Develop a forecast of operating speeds at the most critical curve on the ramp on the basis of actual speeds on existing ramps with similar mainline design speeds, mainline operating speeds, and similar geometrics for the speed-change lane and the portion of the ramp prior to the most critical curve. This forecast should be based on the mainline design and operating speeds, but not on the ramp design speed.
- If the projected ramp operating speed exceeds the design speed, raise the design speed.

If the design speed cannot be raised because of physical or economic constraints, consider speed-control measures such as those discussed next.

On ramps where anticipated operating speeds exceed the maximum feasible design speed, the following speed-control measures should be considered:

- Provide signing with an appropriate advisory speed for the ramp.
- Place the advisory speed signing so that drivers have sufficient distance to slow down between the signing and the most critical curve.
- Increase the length of the deceleration lane or realign the ramp to increase the distance from the gore area to the most critical curve.
- Supplement the standard advisory speed signing to make the signing more conspicuous, to increase the distance from the signing to the most critical curve, and to draw the attention of truck drivers to the signing. These objectives may be accomplished by using more than one ramp speed advisory speed sign, placing ramp speed advisory signing on the mainline highway in advance of the ramp, incorporating an "Exit Speed" panel in the guide signing for the off-ramp, using overhead signing, using a "Truck Speed" advisory sign, and using flashing beacons to call attention to the advisory speed.
- Avoid designs in which the presence of a critical curve on a ramp is not obvious (e.g., where a tight horizontal curve follows a larger-radius curve).
- Consider the use of collector–distributor roads in the interchange. Collector–distributor roads introduce an intermediate-speed roadway between the mainline freeway and the ramp and, thus, may assist in reducing ramp speeds. For example, collector–distributor roads could be appropriate if design constraints necessitated the use of a loop ramp with a 30-mph (48.3-km/h) design speed on a 70-mph (112.7-km/h) freeway.

All of these speed-control measures have been used by highway agencies, but only limited data are available to quantify their effectiveness in reducing speeds. Highway agencies have found that traffic control devices alone are not very effective in reducing vehicle speeds on ramps, but this is not well documented, and further research is needed.

Harwood and Mason indicated that further research is needed in three areas: (1) to identify the potential for skidding and rollover problems on ramp curves designed in accordance with the AASHTO horizontal curve criteria for low-speed design (see Table III-17 of the 1990 *Green Book*) (*1*); (2) to determine the effectiveness of speed-control measures, such as various forms of advisory speed signing, in reducing the travel speeds of passenger cars and trucks on ramp curves; and (3) to develop more effective speed-control methods.

Rajappan and Walton (*252*) assessed the operational impact of longer combination vehicles (LCVs) on the geometry of DIs located along Interstate highways in Texas. The assessment was done by randomly sampling DIs and simulating all possible turn measurements of LCVs at their terminals. The authors concluded that if the LCVs are introduced into the Texas Interstate highway system, the geometry of the existing DI ramp terminals needs to be improved significantly to prevent damage to pavement edges and other roadside appurtenances by the rear wheels of the LCVs. Right turns need greater attention than left turns. The authors also stated that the results of the research could be readily adopted by other states, because the procedure developed in the research can result in conclusions equivalent to those obtained by using other laborious and time-consuming techniques.

Koepke (*253*) reviewed taper and parallel ramp exit and entrance design and concluded that current design elements are acceptable for today's driving conditions. A survey of states found that nearly all agencies use deceleration lane lengths that equal or exceed AASHTO recommendations. The greatest design difference lies in acceleration lane lengths, which in some cases are less than AASHTO recommendations. Research has found that the gore of exit ramps ranks high in the location of freeway accidents, and there are some problems with driver gap acceptance occurring on entrance ramps. Both conditions have been attributed to the assumption that drivers do not know how to properly use, or just do not properly use, speed-change lanes. He recommended additional research in evaluating the operational differences between urban and rural operation, right- and left-side ramps, single- and two-lane ramps, impacts of traffic control devices at freeway/ramp merge or diverge areas, effects of ramp metering on acceleration lane length and operation, and the operation of speed-change lanes at night with or without roadway lighting.

Walker (*254*) reviewed the operational, safety, and capacity aspects of existing two-lane loop ramps. He used the literature along with observations of five two-lane loops to develop the design parameters listed in Figure 40. He concludes that directional or semi-directional ramps are preferred for high-volume, two-lane ramps. They operate at higher speeds and tend to be safer. However, where there is insufficient space or a need to increase the capacity of an existing one-lane loop ramp, the two-lane loop ramp is a reasonable compromise solution. It will provide the required capacity, although not as good service, as the directional ramp. With proper exit, entrance, and speed transition zone design, a loop ramp can carry up to 2,000 vph in a safe and reasonable manner. The capacity will depend on the approach road configuration. Care should also be taken to allow adequate auxiliary lane lengths and acceleration lanes when entering the mainline on an upgrade. The entering 2,000 vph could require a change in the basic number of lanes or an addition of an auxiliary lane, which could have an impact on maintaining lane balance.

Keller (*255*) advocates that ramp geometrics for interchanges be analyzed as a 3-D system to ensure that the facility will function as anticipated. He notes that designers can reduce drivers' stress at interchanges by keeping the alignment simple and direct, maintaining design consistency, providing sight distances greater than minimum stopping sight distances, and using above-minimum design criteria for other geometric elements. He provided the following major principles that should be considered in interchange ramp design:

- The average age of the general population is increasing and therefore the characteristics of the average driver are changing.
- Large truck–trailer combinations are becoming more common and their requirements for proper superelevation rates and longer stopping distances exceed those of the automobile to the extent that previous margins of safety are being eroded.
- Spiral curves provide the most appropriate means to effect superelevation and be certain that the roadway and motorist interact in the manner expected.
- Superelevation rates for ramps used by large trucks should be based on reduced side friction factors.
- The maximum speed differential between adjacent alignment elements should not exceed 10 mph (16 km/h).
- Vertical and horizontal coordination is particularly critical when horizontal curves occur at the end of a downgrade and at the top of a vertical curve, conditions typical of interchange ramp design.

Lomax and Fuhs (*256*) reviewed the geometric design standards and practice of states that operate freeway entrance ramp meters and HOV bypass lanes. They also summarized the 1992 AASHTO *Guide for the Design of High-Occupancy Vehicle Facilities* (*257*). They noted that many states have constructed ramp meters and HOV bypasses in very constrained conditions, and that they have worked well. The retrofit characteristics are frequently a balance between a less-than-desirable level of improvement and no improvement at all. They recommended a more detailed investigation of the compromises inherent in the implementation of ramp meters and HOV bypasses. The investigation needs to identify those approaches that work consistently well and those that, if installed, need more driver information to operate efficiently. They should include such factors as traffic volume, number of lanes, ramp length, merge area, queue storage, signing, signalization and marking, and ramp grade.

Plummer et al. (*258*) identified key design considerations for the at-grade intersection portion of an interchange ramp terminal. They recommended that the following statements be considered in future geometric design policies and research:

- Minimum design speed criteria should be cited for at-grade ramp terminal design on the basis of functional classification of terminal roadway.
- The information shown in Figure 41 could be used to clarify the use of high and low design speeds in relation to the functional classification of the terminal roadway.
- A presentation similar to Table 68 could be used to combine the maximum lateral acceleration values relating to the functional classification of the terminal roadway and respective high- or low-speed criteria.
- Future research should investigate whether the allowable maximum lateral acceleration values have changed since the selection of the original values first published in the *1954 Rural Policy* (*267*).
- The inconsistencies between at-grade intersection sight distance values and the ramp terminal sight distance values should be examined.

In addition, the following statements are recommended for consideration regarding operational analysis of ramp terminals:

Design features of a two-lane loop ramp can be broken into three main categories: the exit, the ramp, and the entrance. The following discussion suggests some design criteria for two-lane loop ramps on the basis of the author's observations and experience. Research is required to assess these criteria.

Exit Design

Exit From Arterial (Parclo A)

Figure A shows a typical Parclo A two-lane ramp exiting from an arterial street. Two lanes are dropped at the exit to increase the capacity of the ramp. The two lanes continue back through the signalized intersection. This allows traffic to line up in proper lanes at the intersection and increase the flow to the ramp. The exit approach could be changed to a three-lane approach with the second lane optional. This would provide better lane balance but would reduce the capacity of the ramp. The selection of either of these designs would depend on the intersection spacing and capacity requirements.

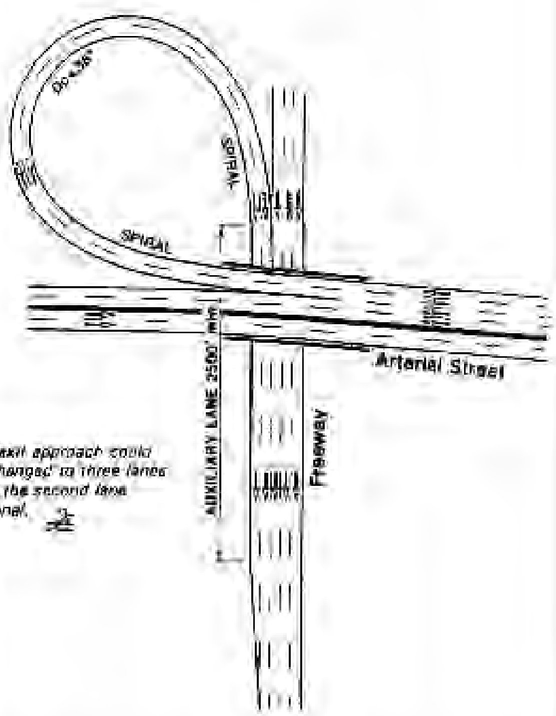


Figure A

Exit From a Freeway (Parclo B)

A single exit before the structure is recommended as shown on Figure B. The two-lanes exit requires an auxiliary lane upstream for 2,500 ft (762.5 m) to develop exit capacity. After the exit, the ramp splits two to one with two continuing to the loop ramp. A 150-ft (45.8 m) radius (25 mph [40.3 km/h]) is normally used for restricted urban conditions. A 230-ft (70.2 m) radius (30 mph[48.3 km/h]) could be used in more open or rural situations to reduce the speed transition by 5 mph (8.1 km/h). Traffic exiting from the freeway will be traveling in the 60- to 70-mph (96.6- to 112.7- km/h) range and will have to decelerate to 25 to 30 mph (40.3 to 48.3 km/h) at the loop ramp. If the configuration is as shown on the dashed lines of Figure B, there will be a tendency for drivers speed up instead of slowing down. They will then have difficulty negotiating the sharp curvature of the loop. When the two lanes of traffic have to go negotiate the sharp curvature of the loop, the design has to provide for as smooth an operation as possible. Any erratic maneuvers are much more hazardous with the two lanes. This means that the geometric design has to provide an alignment that allows drivers to transition to the very slow speed of the ramp. If they enter the loop traveling too fast, they will cause accidents. The curvilinear design (solid line) helps alleviate this problem because drivers will adjust their speed gradually over the whole ramp. A long spiral is preferred at the entrance to the loop ramp to assist in the speed transition. Drivers recognize the curvature of the spiral and can adjust their speed accordingly.

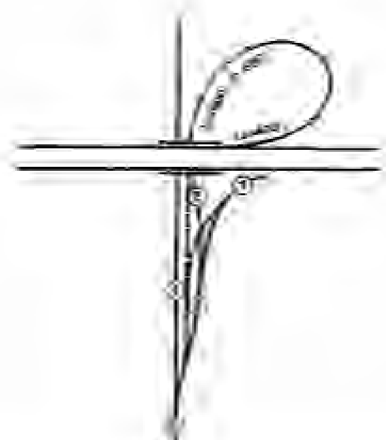


Figure B

FIGURE 40 Design features of two-lane loop ramps (254).

RAMP DESIGN

Figures A and B show a desirable ramp layout with a spiral transition from the exit to the controlling curve of the ramp. This curve is followed by a spiral transition to the entrance area.

The following table gives recommended pavement and shoulder widths for a two-lane loop ramp for 25 mph [150-ft (45.8- m) radius] and 30 mph [230-ft (702.2-m) radius] ramp design speeds. The pavement width is important to provide good lateral clearance between vehicles and to allow for smooth operation. The shoulder widths, 8 ft (2.4 m) right shoulder and 4 ft (1.2 m) left shoulder, allow ample space so that drivers do not feel crowded while making the tight radius turn. These dimensions will allow traffic to flow smoothly and provide the required capacity.

Where the ramp is an appreciable upgrade and there is any chance of trucks' speeds being reduced to a crawl, 0.06 could be considered. This requirement would allow for side slippage of vehicles at slow speeds on ice. The compromise has to be established between the extra super, which is used all the time, and the number of times that ice may be a problem. Values higher than 0.08 in warmer climates would be desirable.

Superelevation will depend on the location. However, a 0.08 maximum would be considered in snow conditions. Vehicles tend to overdrive ramps, and the 0.08 will assist in allowing for this. Normally Parclo A ramps are in a downgrade.

Where the ramp is an appreciable upgrade and there is any chance of trucks' speeds being reduced to a crawl, 0.06 could be considered. This requirement would allow for side slippage of vehicles at slow speeds on ice. The compromise has to be established between the extra super, which is used all the time, and the number of times that ice may be a problem. Values higher than 0.08 in warmer climates would be desirable.

Entrance Design

The standard two-lane entrance design from AASHTO should be used, which will allow for an auxillary lane of 2,500 ft (762.5 m) for turning volumes of 1,500 to 2,000 vph and 3,000 ft (915.0 m) for volumes in excess of 2,000 vph. Whereas a single-lane ramp can accommodate up to 1,500 vph, it is unlikely that a single exit or entrance can, unless a line is dropped at the exit or added at the entrance. A lane drop at the exit would not provide good lane balance, and therefore a two-lane exit is more desirable for volumes over 1,000 vph. A single lane at the entrance would be acceptable.

Traffic Condition	Left Shoulder ft (m)	Pavement Width ft (m)	Right Shoulder ft (m)	Radius ft (m)
A	4 (1.2)	26 (7.9)	8 (2.4)	150–230 (45.8–70.2)
B	4 (1.2)	28 (8.5)	8 (2.4)	150–230 (45.8–70.2)
C	4 (1.2)	30 (9.2)	8 (2.4)	150–230 (45.8–70.2)

FIGURE 40 (*Continued*).

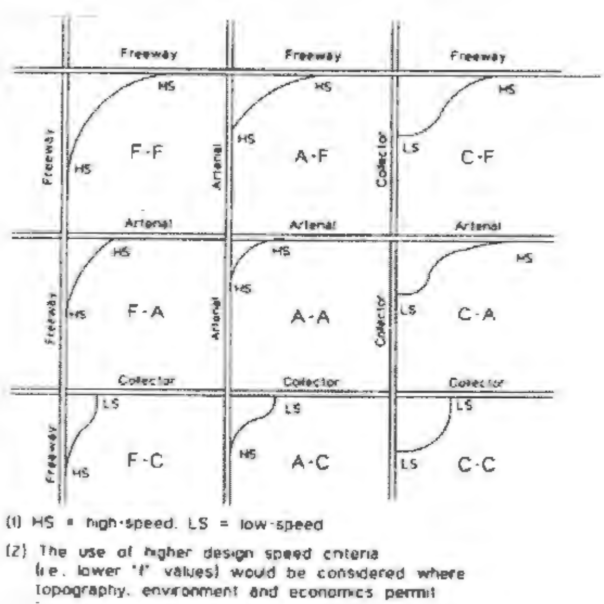


FIGURE 41 Minimum design speed criteria for ramp terminals (*258*).

- Ramp terminals designed according to high-speed design criteria should be analyzed according to ramp junction capacity analysis procedures.
- At-grade ramp terminals designed according to low-speed design criteria should be analyzed according to at-grade intersection capacity analysis procedures.
- The capacity estimation procedures in the current *Highway Capacity Manual* (*13*) should be modified to reflect the higher volumes that are frequently observed at these types of facilities.

USER NEEDS

Knoblauch et al. (*259*) identified characteristics of older drivers that affect their ability to drive on freeways and recommended research to develop guidelines for countermeasures to accommodate their needs. The recommendations were developed based on literature review, focus group discussions, computerized accident analysis, hard-copy accident analysis, AARP survey, and travel diary study. Table 69 lists their recommendations.

TABLE 68
MAXIMUM LATERAL ACCELERATION VALUES (SIDE FRICTION FACTORS, *F*) FOR HORIZONTAL CURVE DESIGN AT RAMP TERMINALS (*258*)

Functional Classification[a]	Criteria	Minimum Turning Roadways 20 mph (32.2 km/h)	30 mph (48.3 km/h)	40 mph (64.4 km/h)	High-Speed Merge/Diverge 50 mph (80.5 km/h)	60 mph (96.6 km/h)	70 mph (112.7 km/h)
Freeway	HS	NA	0.16	0.15	0.14	0.12	0.10
	LS	NA	NA	NA	NA	NA	NA
Arterial	HS	0.17	0.16	0.15	0.12	0.12	0.10
	LS	0.27	0.23	0.16	NA	NA	NA
Collector	HS	0.17	0.16	0.15	0.14	NA	NA
	LS	0.27	0.23	0.16	NA	NA	NA
Local	HS	0.17	0.16	0.15	NA	NA	NA
	LS	0.27	0.23	0.16	NA	NA	NA

Notes: For horizontal curve departing or entering the adjacent roadway. HS = high-speed criteria (AASHTO 1990 *Green Book*, p. 154, Table III-6); LS = low-speed criteria (AASHTO 1990 *Green Book*, p. 197, Table III-17). NA = not available.
[a]From and/or to which the ramp proper is connecting.

TABLE 69
RECOMMENDED RESEARCH ON OLDER DRIVERS (*259*)

Research Area	Problem(s)	Needed Research
Navigation/Wayfinding	Difficulties navigating. Overinvolvement in accidents in unfamiliar areas.	Further define problem. Develop and test alternative traffic control device designs.
Freeway Ramp Merging	Merging onto mainline from ramps.	Identify ramp geometrics and mainline characteristics that contribute to the problem. Develop and test alternative designs.
Freeway Transition Areas	Exit ramps, bifurcations, and lane drops.	Identify geometric features and traffic control devices to minimize problems in transition areas.
Illumination Requirements	Nighttime driving. Reduced driving at night.	Identify critical factors associated with highway lighting, e.g., placement, amount of lighting.
Speed/Lane Selection	Inappropriate lane selection. Inappropriate speed selection.	Identify relevant design parameters: horizontal/vertical curvature; lane, roadway, and shoulder width; median type and proximity; guardrail type and proximity.
Construction Areas	A major concern. A reason for avoiding freeways.	Identify characteristics of construction areas that are troublesome to older drivers. Develop and test treatments to improve older driver performance in CZs.
Fatigue/Medication	Fatigue is a major factor in single-vehicle accidents. Many survey respondents indicated they were often fatigued while driving.	Identify roadway characteristics (geometrics, delineation, lighting) associated with fatigue and fatigue-related crashes.
Lane Changing/Passing Behavior	Lane-changing-related crashes. Problems reported with passing/lane changing.	Conduct detailed behavioral analysis of lane changing and passing behavior. Determine adequacy of exit signing and advanced exit signing relative to time needed to complete passing maneuver and/or change lanes to exit.
Roadway Delineation	Heavy reliance on delineation, RPM's, and postmounted delineators. Run-off-roadway and lane-changing accidents may be related to poor delineation.	Determine optimal delineation width and reflectance RPM and post-mounted delineator spacing, etc. Wet/nighttime performance is especially critical.
Roadway Signing	Strong preference for overhead signing and for listing multiple exits on advance warning systems.	Determine readability, legibility, advantages and disadvantages of overhead vs. shoulder-mounted signing. Determine optimal message content (length and format) of advance exit signing.
Rest Areas	The older drivers use exiting rest areas and would like more of them. Fatigue-related accidents might be reduced if more rest areas were provided.	Determine optimal rest area spacing and characteristics (services, lighting, security, etc.).
Toll Plaza Design	Merging with other vehicles to get in line and merging with other vehicles leaving the plaza.	Determine factors that affect merging behavior when approaching and leaving toll booths (delineation, signing, illumination, etc.). Optimize the most salient factors for older drivers.
Congestion-Related Accidents	24 percent of the older driver multiple-vehicle accidents involve striking stopped or slowing vehicles.	Determine effect of both active and passive advance warning signs for "congestion ahead" situations. Determine optimal type, placement, and wording.
Glare	Glare from oncoming vehicles, following vehicles, and roadway lighting.	Identify nature of glare problems. Evaluate effect of median barriers, glare screens, and overhead lighting (placement and intensity) on older drivers.

Notes: CZ = construction zone; RPM = raised pavement markers.

The *Older Driver Highway Design Handbook* (*7*) provides specific recommendations for highway design elements in the following four areas for interchanges: exit signing and exit ramp gore delineation, acceleration/deceleration lane design features, fixed lighting installation, and traffic control devices for prohibited movements on freeway ramps. The following list presents examples of the recommendations included in the *Handbook*.

A. Exit Signing and Exit Ramp Gore Delineation

- The calculation of letter size requirements for exit signing bases on an assumption of not more than 33 ft (10 m) of legibility distance for each 1 in. (25 mm) of letter height is recommended for new or reconstructed installations and at the time of sign replacement.
- It is recommended that the *Manual on Uniform Traffic Control Devices* (MUTCD) (Federal Highway Administration 1988) requirements for multiple advance signing upstream of major and intermediate interchanges (section 2E-26) be extended to minor interchanges as well.
- A modification of diagrammatic guide signing displayed in the MUTCD (Figure 2-30), such that the number of arrow shafts appearing on the sign matches the number of lanes on the roadway at the sign's location, is recommended for new or reconstructed installations.

B. Acceleration/Deceleration Lane Design Features

- It is recommended that acceleration lane lengths be determined using the higher of AASHTO (1994) Table X-4 speed change lane criteria or NCHRP 3-35 (Reilly et al. 1998) values for a given set of operational and geometric conditions, and assuming a 40 mph (64 km/h) ramp speed at the beginning of the gap search and acceptance process.
- A parallel versus a taper design for entrance ramp geometry is recommended.
- It is recommended that post-mounted delineators and/or chevrons be applied to delineate the controlling curvature on exit ramp deceleration lanes.
- It is recommended that AASHTO (1994) decision sight distance values be consistently applied in locating ramp exits downstream from sight-restricting vertical or horizontal curvature on the mainline [instead of locating ramps based on stopping sight distance (SSD) or modified SSD formulas].

C. Fixed Lighting Installations

- Incomplete interchange lighting (CIL) is the preferred practice, but where a CIL system is not feasible to implement, a partial interchange lighting (PIL) system comprised of two high-mast installations per ramp—one fixture on the inner ramp curve near the gore and one fixture on the outer curve of the ramp, midway through the controlling curvature—is recommended.

D. Traffic Control Devices for Prohibited Movements on Freeway Ramps

- The consistent use of 48 in. x 30 in. (1200 mm x 750 mm) FREEWAY ENTRANCE signs for positive guidance as described as an option in section 2E-40 of the MUTCD (Federal Highway Administration 1988), using a minimum letter height of 8 in. (200 mm) with series D or wider font, is recommended.
- Where adjacent entrance and exit ramps intersect with a crossroad, the use of a median separator is recommended, with the nose of the separator delineated with yellow reflectorized paint and extending as close to the crossroad as practical without obstructing the turning path of vehicles. In addition, it is recommended that a KEEP RIGHT (R4-7a) sign be posted on the median separator nose.
- Where DO NOT ENTER (R5-1) and WRONG WAY (R5-9) signs are placed, in accordance with sections 2A-31 and 2E-40 of the MUTCD, a minimum size for R5-1 of 36 in. × 36 in. (900 mm × 900 mm) and for R5-9 of 48 in. × 32 in. (1200 mm × 800 mm) is recommended, with corresponding increases in letter sizes, and the use of high-intensity sheeting. In addition, a mounting height (from the pavement to the bottom of the bottom sign) of between 18 and 36 in. (450 mm and 900 mm) is recommended, using the lowest value for this range that is practical when the presence of snow or other obstructions is taken into consideration.
- The application of 24-ft (7.3-m) wrong-way arrow pavement markings (see MUTCD, section 2B-20) near the terminus on all exit ramps, accompanied by red raised pavement markers facing wrong-way traffic, is recommended.

Lunenfeld (*260*) examined the human factors issues associated with interchange design features. The following lists the principles and an accompanying discussion.

- Design for Drivers and Target Populations—Information at an interchange should be presented in nontechnical terms because drivers may not understand engineering concepts. It should also be determined whether there are target groups whose needs must be addressed. These groups may be older drivers with vision problems or truck drivers negotiating sharp ramps.
- Be Responsive to Task Demands and Driver Attributes—Highway information should convey the operating conditions of interchanges, be responsive to the

task demands imposed on the driver by interchange design and geometry (particularly when there are time pressures caused by traffic), and be sensitive to driver sensory-motor attributes. Drivers may become overloaded when they have to process too many sources of information, or when an information source has too much information content. Overloaded drivers may become confused or miss important information sources.

- Satisfy All Information Needs—All information needs relative to all aspects of the driving task at the interchange should be satisfied. Speed and path information should always be available. Information needs pertaining to routes, destinations, directions, and services should be displayed when needed, where required, and in a form best suited to the driver and task.
- Maintain Interchange Design and Information System Compatibility—Because drivers formulate driving strategies on the basis of their perception of the design and operations of an interchange, incompatible information displays will lose credibility and may lead to confusion. A determination should be made on how interchange designs and information treatments appear to drivers and whether they are compatible and do not violate expectancies. In the design stage, models or computer simulations could be used to make this determination and to ensure compatibility.
- Avoid Surprises—Driver performance is enhanced when forward sight distance provides a clear, unobstructed view of an interchange, its traffic, and its traffic control devices. Problems often occur when drivers are surprised by unexpected or unseen features. If any aspects of the interchange could surprise drivers, advanced warning should be provided.
- Eliminate Information-Related Error Sources—Information-related error sources should be eliminated. These sources include missing information; information obscured by foliage, structures, earth berms, dirt, snow, or the like; misplaced information (not in a driver's field of view); devices too close to a choice point; and obsolete, nonstandard, ambiguous, or confusing messages.
- Resolve Conflicts When Information Sources Compete—When information sources compete for a driver's limited processing capacity (generally 5 to 9 sources or 2 to 3 bits of information), or when there is a chance of overload, a determination should be made as to what information sources should be displayed, and which should be spread out, moved, or removed. Generally, guidance information relating to speed and path takes precedence over navigation information relating to direction.
- Use Spreading—Spreading reduces the chance for overload at high-processing-demand locations by moving less important information sources upstream or downstream, thereby reducing the processing load.
- Use Repetition for Interchange Information Treatments—If a time greater than 30 sec to 2 min, a driver's short-term memory span, intervenes between the receipt of advance interchange information and the exit ramp, drivers may forget the message. Repeating the information one or more times will help drivers remember and act on it. Repetition is also useful if an information display might be blocked by foliage or trucks.
- Use All Available Navigation Aids and Treatments—Appropriate navigation aids should be used. These aids include overhead signs that can be seen over trucks, oversized route guidance signs to help drivers at choice points, trailblazers to freeways and interchanges, real-time changeable message signs to warn of incidents and help manage congestion, and highway advisory radio, which transmits information into a road-user's vehicle.

Lunenfeld emphasized that "engineers and designers should be aware of and account for all of them and bear in mind that their efforts are first and foremost to aid the driver."

ACCIDENTS

Twomey et al. (*261,262*) critically reviewed and summarized past safety research on interchanges. Some of their findings include the following:

- Interchange ramps should be designed with flat horizontal curves (except in rural areas), and the maximum degree of curvature for a given design speed and superelevation should be avoided.
- Sharp curves at the end of ramps and sudden changes from straight alignment to sharp curves should be avoided.
- Cloverleaf ramps, left-side ramps, and scissor ramps should be avoided where possible.
- Urban interchanges have much higher accident rates than rural interchanges (Table 70). The relative safety of entrance and exit terminals is enhanced with geometric designs that provide 800 ft (244.0 m) or longer acceleration or auxiliary lanes. Deceleration lanes 900 ft (274.5 m) or longer reduce traffic friction on the through lanes and account for reduced accident rates. Geometric design for weaving maneuvers should provide weaving sections that are at least 800 ft (244.0 m) long.
- Interchange accident rates have been shown to increase as interchange spacing decreased in urban areas.

An FHWA research project conducted by Bauer and Harwood (*263*) developed statistical models to define the

TABLE 70
ACCIDENT RATE BY INTERCHANGE UNIT AND AREA TYPE (*261*)

Interchange Unit	Rural			Urban		
	Vehicle-Miles (100 million)	No.of Accidents	Accident Rate[a]	Vehicle-Miles (100 million)	No. of Accidents	Accident Rate[a]
Deceleration Lane	2.51	348	137	5.83	1,089	186
Exit Ramp	0.57	199	346	1.48	546	370
Area Between Speed Change Lanes	6.52	554	85	11.87	1,982	167
Entrance Ramp	0.59	95	161	1.61	1,159	719
Acceleration Lane	3.68	280	76	8.40	1,461	174
Acceleration–Deceleration Lane	0.49	87	116	2.45	555	227
Total	14.36	1,563	109[b]	31.64	6,792	214[b]

Notes: 1 mi = 1.61 km.
[a]Accidents per 100 million vehicle-miles.
[b]Average accident rate.

relationship between traffic accidents and highway geometric design elements and traffic volumes for interchange ramps and speed-change lanes. The regression models developed, based on the negative binomial distribution, explained between 10 and 42 percent of the variability in the accident data, with the negative binomial distribution providing a poor to moderate fit to the data. However, most of that variability was explained by ramp AADT. Other variables found to be significant in some models included mainline freeway AADT, area type (rural/urban), ramp type (on/off), ramp configuration, and combined length of ramp and speed-change lane. The best models for predicting accident frequencies were those obtained when modeling the combined accident frequency for an entire ramp, together with its adjacent speed-change lanes. These models provided a better fit than separate models for ramps and speed-change lanes.

In California, ramp accidents accounted for 2.8 percent of the accidents on rural highways and 18.4 percent of accidents on urban highways (*264*). Accident rates were analyzed using analysis of variance and analysis of covariance methods. The analyses were intended to look at the systematic difference in accident rates between ramps of different design, stratified by whether a ramp was in a rural or urban area, as well as whether it was an on-ramp or an off-ramp. Scissor ramps, rest area ramps, and slip ramp configurations were the most common among the ramp samples with high accident rates. (A scissor ramp is a direct one-way through ramp with traffic to or from a local two-way facility, where local traffic can cross in front of the ramp traffic in generally a scissors movement, which can be an angle of 90 degrees or less.)

Accident rates for on-ramps were consistently lower than those for off-ramps. The author noted that the independent variables most often found by studies to be statistically significant were ramp AADT, mainline freeway AADT, area type (rural/urban), ramp type (off/on), ramp configuration, length of speed-change lane, and ramp length. The general consensus of many studies is that traffic volume is the strongest predictor of accidents on ramps. The studies did not suggest that geometric variables are unimportant, but that geometric factors are generally improved to a point where further variance in geometric features has little influence on accidents.

CHAPTER SEVEN

FINDINGS AND CONCLUSIONS

The review of the literature published in the 1990s identified a number of key findings that could have an impact on current practice or methodology. A limitation to the potential application of the research was the sheer volume of information that had been published. The NCHRP synthesis project funded a study to develop a synthesis that would summarize the geometric design research, particularly research with safety and operational implications. This chapter provides a summary of the key findings from the literature published in the 1990s.

When a research study provided a single or only a few key findings, this information is provided within the sections below. In most cases, however, the research studies produced several findings. To keep this chapter to a size conducive to quick reading, these studies are noted, with presentation of the studies' multiple findings (including tables or figures where appropriate) provided in chapters 2–6. Information on the source of the findings is provided in these chapters as well as in this Findings and Conclusions chapter so that the reader can consult the original document for additional details.

This chapter also includes five lists that summarize changes, additions, or reference documents that should be considered during a revision of a design manual and covers the following areas: design controls and criteria, elements of design, cross sections, intersections, and interchanges.

DESIGN CONTROLS AND CRITERIA

Design controls and criteria provide the designer with tools that help determine whether designs are acceptable for use. By providing specific measures used for comparison to reference values, controls and criteria help the designer to provide designs that meet the needs of the user. Several recent studies have developed new measures and additional comparison values for use by the engineer. The following list cites issues and reference documents to be considered in future editions of design manuals:

- Reference the "Building a True Community" report by the Public Rights-of-Way Access Advisory Committee or consider incorporating some of the findings (*35*).
- Reference the *Older Driver Highway Design Handbook* (*7*) or consider incorporating some of the findings and recommendations.
- Reference the new edition of the *Highway Capacity Manual* (published in 2000) (*13*), the *Guide for the Development of Bicycle Facilities* (*36*), the ITE *Design and Safety of Pedestrian Facilities* (*33*), appropriate documents on safety (*37–44*), and the Interactive Highway Safety Design Model (*45*).
- Determine the needed revisions to the Access Control section using information provided in a number of references including material in *NCHRP Report 348* (*14*), *NCHRP Report 420* (*15*), and *TRB Research Circular 456* (*17*).
- Consider Pietrucha and Opiela's (*34*) suggestions on revising the *Green Book* to encourage designers to "think about the pedestrian."
- The concept of design consistency should be discussed more extensively within the *Green Book*. Alternative methods for achieving design consistency should be presented. Methods include a speed profile model (*58*) and a work-load model (*59*).

Gattis and Howard (*3*) developed design vehicle characteristics for large- or full-size school buses. Harkey et al. (*4*) encouraged additional research to evaluate how longer-combination vehicles operate. Harwood et al. (*6*) determined the distribution of the following geometric criteria for 436 interchanges and 379 at-grade intersections located in 9 states: radii of horizontal curves on mainline roadways, grades on mainline roadways, radii of horizontal curves on interchange ramps, curb return radii at ramp terminals, and curb return radii for at-grade intersections. Using 5 years of rural Interstate highway data from Utah, Miaou and Lum (*5*) developed expected reductions in truck accident involvement rates caused by an improvement in one geometric design element and two geometric design elements.

The FHWA sponsored a major study that examined older driver needs. One result of the study was the *Older Driver Highway Design Handbook* (*7*), which contains several specific recommendations for accommodating older drivers. It is intended to supplement standard design manuals for practitioners.

Fitzpatrick et al. (*11*) reviewed the relationship between design speed, operating speed, and posted speed and found that DOT officials are concerned with the potential liability; however, only a few of the respondents to surveys and interviews actually experienced a lawsuit relevant to the design speed-posted speed issue. The respondents indicated that the definition of design speed should be changed.

A new edition of the *Highway Capacity Manual* was published in 2000 (*13*). The most recent and comprehensive document related to the design of facilities for bicycles is AASHTO's *Guide for the Development of Bicycle Facilities* (*36*). During the 1990s, several general safety documents were published including *Accident Mitigation Guide for Congested Rural Two-Lane Highways* (*37*), *Roadside Design Guide* (*40*), and others (*38, 39, 41–44*). Also during the 1990s, the FHWA worked on developing the Interactive Highway Safety Design Model (IHSDM) in an attempt to marshal available knowledge about safety into a more useful form for highway planners and designers (*45*).

During the 1990s, several studies reviewed access control issues. *NCHRP Report 348* (*14*) reviewed the overall concept of access management and current practices and set forth basic policy, planning, and design guidelines. *NCHRP Report 420* (*15*) reviewed the impacts of access management techniques on travel time and accidents and provided several tables that can be used to appraise access management. The *TRB Research Circular 456*, *Driveway and Street Intersection Spacing* (*17*), compiled the contemporary practice that illustrates the basic considerations for spacing standards and guidelines and that describes current state, county, and local spacing requirements. It also provided optimum intersection spacing for signalized and unsignalized intersections. Several other papers provided recommended signal spacings (*22,23*), discussed the benefits of access management on reducing accident experience (*24–26*), presented major issues associated with access control (*18*), and determined the needed spacing of driveways near intersections (*27*).

Pietrucha and Opiela (*34*) examined the *Green Book* from a pedestrian design perspective to determine the adequacy of highway design standards in considering the pedestrian, the appropriateness of current design treatments, the compatibility of pedestrian facility designs and highway facility designs, and the effectiveness of the various treatments. They recommended changes and short additions to several areas. Cheng (*32*) investigated the trend of Utah's pedestrian accident rate. He notes that because most pedestrian accidents are caused by pedestrian error, more emphasis must be placed on modifying the training and behavior of pedestrians in crossing techniques, particularly for school-age pedestrians. In 1998, ITE published *Design and Safety of Pedestrian Facilities,* which presents guidelines for the design of pedestrian facilities in order to provide safe and efficient opportunities for people walking near streets and highways (*33*). AASHTO is sponsoring the development of a pedestrian report through NCHRP Project 20-7.

The Public Rights-of-Way Access Advisory Committee (PROWAAC) was convened by the U.S. Architectural and Transportation Barriers Compliance Board (the Access Board) to address access to public rights-of-way for people with disabilities. Near the end of the preparation of this synthesis, the PROWAAC presented the completed report ("Building a True Community") to the U.S. Access Board (*35*). The report (available online at www.access-board.gov) discusses many issues including traffic calming, roundabouts, and overall design for pedestrians. It provides a new national set of guidelines that define the details necessary to make the streetscapes in public rights-of-way accessible to all users. They note that ". . . the guidelines proposed do not call for a minor adjustment here and there, they ask for a dramatic change from the way public rights-of-way have been designed in the past." The report also states, "It is important to understand that the recommended standards, if adopted, will apply whenever new streets are created and whenever existing streets are reconstructed or otherwise altered in ways that affect their usability by pedestrians." Implementation of these recommendations will not require jurisdictions to rebuild existing streets solely to meet these standards.

Several researchers have developed approaches for obtaining design consistency on rural two-lane highways (*46–59*). The most promising of the approaches appears to be a speed prediction model (*58*). In this type of approach, a roadway design (or existing alignment) is analyzed for its effects on speed. The geometric features that cause large changes in speed are identified.

ELEMENTS OF DESIGN

Individual design elements have been subjected to a number of evaluations, with many focused on roadway curvature, sight distance, and passing behavior. These issues encompass and affect the designer's selection of roadway alignments and the basic layout of the transportation network. The following list presents issues for consideration in future editions of design manuals:

- Incorporate findings from *NCHRP Report 400* on stopping sight distance (SSD), including changes in driver eye height and object height (*74*).
- Add a comment that on horizontal curves with lower design speeds, the most unstable trucks can roll over when traveling as little as 8 to 16 km/h (5 to 10 mph) above the design speed (*83*).
- Comment that for design speeds of 16 to 32 km/hr (10 to 20 mph), minimum-radius horizontal curves may not provide adequate margins of safety for trucks with poor tires on a poor wet pavement or for trucks with low rollover thresholds. Revision of the criteria in *Green Book*, Table III-17, should be considered,

especially for locations with substantial truck volumes. This same concern is applicable to the horizontal curve design criteria for low-speed urban streets based on *Green Book*, Table III-6 (*83*).

- State explicitly that minimum radii smaller than those shown in *Green Book*, Table III-17, should not be used, even when they appear justified by above-minimum superelevation rates (*83*).
- Incorporate changes to the superelevation section based on findings from *NCHRP Report 439* (*90*).
- Introduce the concept of good, fair, and poor design practices based on the speed differential between consecutive sections or between the design speed and the operating speed (*97*).
- Add comment on the use of zero-length and minimum-length vertical curves (*98*).
- Comment that side road intersections are to be avoided within a passing lane section (especially if high volume) and within lane drops and lane additions. They should be located near the middle if low volume (*109*).
- Review the various methods available for determining passing sight distance and determine if the method and/or values presented within the *Green Book* should be revised (*109,110,112,115*). Consider modifying passing sight distance to consider the situation when a passenger car is passing a truck or a bus (*110*).
- Add comments on alternative methods for measuring passing sight distance (*113*).
- Investigate whether a delay warrant should be used for climbing lanes and if Homburger's recommendations on climbing lane length and location of terminals should be adopted (*119,120*).

New values for SSD were recommended in *NCHRP Report 400* (*74*). These values have been accepted for the 2001 edition of the *Green Book*. Accompanying the new values were changes in several of the assumptions used in developing SSD or using the SSD values. For example, one initial speed rather than having minimum and desirable values was recommended. Friction was replaced with driver deceleration, driver eye height was changed to 1080 mm and the object height was raised from 6 in. (150 mm) to 24 in. (600 mm). *NCHRP Report 400*, along with another study, supported the use of the 2.5-sec perception–reaction time.

During the 1990s, Gattis (*80*) and Easa (*81,82*) presented sight distances for unique situations. Gattis (*80*) discussed the head-on sight distance needs for when traffic moving in opposite directions operates in one lane, as happens on some residential streets. Easa (*81*) looked at compound horizontal curve sight distance for an obstacle located within the sharper or flatter arc of a compound curve. Easa (*82*) also investigated sight distance needs for reverse horizontal and vertical curves.

Harwood and Mason (*83*) conducted an evaluation of horizontal curve design policy and made conclusions for both high-speed and low-speed designs. For high-speed design, they concluded that there does not appear to be a need to modify existing criteria for determining the radii and superelevation of horizontal curves in *Green Book,* Table III-6. Existing design policies provide adequate margins of safety against skidding and rollover by both passenger cars and trucks as long as the design speed of the curve is selected realistically. Special care should be taken for curves with design speeds of 48 km/h (30 mph) or less to ensure that the selected design speed will not be exceeded, particularly by trucks. For low-speed design, minimum-radius horizontal curves designed in accordance with the low-speed criteria in *Green Book*, Table III-17, generally provide adequate margins of safety against skidding and rollover for passenger cars traveling at the design speed. For design speeds of 16 to 32 km/h (10 to 20 mph), minimum-radius horizontal curves may not provide adequate margins of safety for trucks with poor tires on a poor wet pavement or for trucks with low rollover thresholds. They recommended that the *Green Book* be revised to reflect their findings on truck performance on low-speed horizontal curves.

Reviews (*83,84*) of the benefits of different methods of achieving superelevation found that transitions designed according to the two-thirds/one-third rule proved to be acceptable. A comprehensive review of superelevation was conducted and reported in *NCHRP Report 439* (*90*). The report includes detailed recommendations for changes to the *Green Book* to simplify the design of curves and to result in more consistent curve design throughout the United States.

Several studies examined the relationship between horizontal curve features and accident experience. A key finding from several studies was that higher accident rates are associated with higher volumes and sharper curves (*55,91,92,96*). Potential improvements include curve flattening, widening lanes, and improving superelevation. An informational guide was developed to assist with the design of horizontal curves on new highway sections and with the reconstruction and upgrading of existing curves on two-lane rural roads (*93*). The guide also gives a step-by-step procedure for computing expected benefits and costs for a variety of curve improvements. Lamm et al. (*97*) developed a procedure for evaluating the horizontal alignment of two-lane rural roads that provides a level of design practice (good, fair, or poor) based on the speed differential between operating speeds on consecutive sections, speed differential between design speed and operating speed for a section, and the differential between side friction assumed and demanded for a section.

Wooldridge et al. (*98*) evaluated the use of zero-length and minimum-length vertical curves and developed recommendations on their use. The relationship of accidents to limited sight distance crest vertical curves was investigated by Fitzpatrick et al. (*99*). Crash rates on limited sight distance curves were found to be similar to the crash rates on all two-lane rural highways. Crash rates for large trucks and older drivers were also similar for limited sight distance curves and all two-lane rural highways. Thus, for the range of conditions studied (e.g., no major intersections within study area, etc.), limited SSD does not appear to cause a safety problem.

Several authors investigated issues associated with passing on two-lane rural highways. Mutabazi et al. (*109*) investigated the safety of different locations of crossroad intersections relative to passing lanes and the operational efficiency of different passing lane configurations. They found that different passing lane configurations appear to differ only marginally. They also developed recommendations for the location of side road intersections.

Polus et al. (*110*) collected data on 1,500 passing maneuvers and compared their findings to the 1994 *Green Book* (*2*). They found that the overall sight distance is adequate when a car passes a car; however, it is not sufficient for a car passing a truck. Although the total sight distances are similar between the *Green Book* model and the model developed by Polus et al. (*110*), the individual components are considerably different. Because the study found that about one-half of all passing maneuvers involved a truck being overtaken, it was recommended that consideration of passenger cars passing trucks and buses is needed.

Sparks et al. (*112*) also reached the conclusion that the critical passing maneuver (e.g., car passing a car or car passing a truck) to be considered in highway design needs to be resolved. Glennon (*115*), however, stated that the effect of truck length on passing sight distance is not as dramatic as reported by others. He also commented that very short passing zone lengths, such as the 400-ft default length allowed by the *Manual on Uniform Traffic Control Devices*, are not appropriate for safe highway operations. He provided recommended minimum passing zone lengths.

Brown and Hummer (*113*) evaluated methods for measuring passing sight distance on two-lane, two-way highways. Harwood et al. (*116*) presented information on effectively locating, designing, signing, and marking passing lanes to improve traffic operations. Most of the information was from a 1987 FHWA report, *Low-Cost Methods for Improving Traffic Operations on Two-Lane Highways: Informational Guide* (*117*).

Wolhuter and Polus (*119*) suggested that delay be used as a warrant for the installation of climbing lanes. If their warrant were adopted, it would result in a decrease in climbing lanes warranted on flatter grades and a similar increase on steeper grades. Homburger (*120*) investigated traffic flow at the upper end of a climbing lane and developed recommendations on a climbing lane length and location of the terminal.

CROSS SECTIONS

A number of studies have examined issues related to roadway cross sections. Ranging from studies of the effects of reallocations of available roadway width to reviews of the safety effects of individual elements, these studies have developed several recommendations along with discussions of tradeoffs between design alternatives. A list of issues or references to be reviewed for a design manual revision follows:

- Discuss the tradeoffs of using shoulders and/or narrowing travel lanes on freeways (*129*).
- Discuss the techniques available for selecting alternative median treatments (*16*) and tradeoffs between median types (*16,133–137*).
- Present tradeoffs for alternative uses of a cross section (*138–143*).
- Add reference to *NCHRP Report 413* (*143*) and *NCHRP Report 414* (*144*) on high-occupancy vehicle (HOV) lanes.
- Add comment about using 12-ft lanes and 3-ft paved shoulders on rural roadways carrying truck traffic consisting of 96-in. or 102-in. wide trucks (*145*).
- Add appropriate comments regarding design alternatives for lower speed roadways (*147–150*).
- Add appropriate comments regarding the use of curb and gutter on suburban highways (*151*).
- Add appropriate comments regarding the relationship between safety and cross-sectional elements (*153–164*).

A 1995 NCHRP study (*129*) confirmed that using shoulders as travel lanes and/or narrowing travel lanes on a freeway can be effective to increase capacity in congested urban corridors. However, findings indicate that in many instances there may be measurable negative impacts to the overall safety performance of the corridor. These strategies should be reserved for use as techniques for congestion relief and not as a means to widen facilities for extended lengths. An earlier study (*130*) that investigated the effects on accidents also supported the finding that using shoulders could result in improved operations and improved safety within the section.

A 1997 NCHRP study (*16*) evaluated the operational and safety effects of three alternative median treatments. A set of guidelines was developed for the selection of midsignal median treatment. The guidelines were based on field data, safety, and road-user benefits. Several other projects also investigated the performance of various median treatments. The studies found that undivided roadways have higher vehicle accident rates (*133,136*) and higher pedestrian accident rates (*135,136*) than divided facilities [two-way left-turn lane (TWLTL) or raised median]. A 1994 study (*135*) found that the replacement of a TWLTL with a raised median resulted in a drop in accidents for the section, and another 1994 study also found that the pedestrian accident rate for raised medians was lower than for TWLTL and undivided (*136*).

Harwood (*138*) determined the effectiveness of various alternative strategies for reallocating the use of street width on urban arterials without changing the total curb-to-curb width in *NCHRP Report 330*. He found that the safety of urban arterials could be improved by implementing strategies that involve the use of narrower lanes. Knapp et al. (*137*) used past research and case study experience to examine the issues of reducing the number of lanes from a four-lane undivided section to a three-lane section to improve roadway operations. A study (*142*) that examined the use of curb on high-speed suburban multilane highways found that when driveway density is low and traffic volume is high, curb and gutter may not be the best option, as it may increase accident rates. On roadways with high driveway densities, however, curb and gutter may help the road to operate more safely.

A study on bicycle lanes versus wide curb lanes found that significant differences in operational behavior and conflicts occur between the two treatments; however, it varied depending upon the behavior being analyzed (*139*). The overall conclusion from the study is that both bike lanes and wide curb lane facilities can and should be used to improve riding conditions for bicyclists. Three documents were produced from the study: a final report (*139*) containing a complete discussion of the research method, data collection procedures, and data analysis; an implementation manual (*140*); and a guidebook (*141*) on innovative bicycle accommodations. Other advice related to the design of facilities for bicycles is contained in AASHTO's *Guide for the Development of Bicycle Facilities* (*36*).

The most recent work on HOV lanes is contained in *NCHRP Report 413* (*143*) and *NCHRP Report 414* (*144*). *NCHRP Report 413* (45 pages) documents gaps and weaknesses in the current practices for developing or expanding HOV systems. It presents an implementation plan for transferring the completed *HOV Systems Manual* into practice. *NCHRP Report 414* (721 pages) is a comprehensive and detailed *HOV Systems Manual* that incorporates current guidelines and practices.

A study on wider trucks found that they have significantly higher rates of edgeline encroachments and their drivers tend to drive closer to the centerline than drivers of narrower trucks (*145*). The authors recommended that 12-ft lanes and a minimum of 3 ft of paved shoulders should be considered on rural roadways carrying truck traffic consisting of either 96-in. or 102-in. wide trucks. Harwood et al. (*146*) estimated the extent of geometric design improvements and the costs of those improvements to accommodate particular truck configurations on individual roadway networks. Substantial costs would be required to accommodate potential future trucks on the existing roadway system. These costs are sensitive to the size of the truck and the extent of the roadway system considered.

Geometric design dimensions for several traffic calming measures used on low-speed roadways are provided in *Traffic Calming State of the Practice* (*147*). Gattis and Watts (*150*) investigated the relationships among urban street function (arterial and local), width, and resulting speed. The findings suggested that street width may play a small role in vehicle speed, but other factors such as trip function may be more significant determinants of the average and 85th percentile vehicle speeds. Fwa and Liaw (*148*) developed an approach to selecting the geometric dimensions of a hump based upon the design 85th percentile hump-crossing speed and a peak vertical acceleration that governs drivers' choice of hump-crossing speeds. The authors cautioned that additional verification and more evidence are needed before the Singapore-based data for speed-control hump design are applied in other regions. Polus and Craus (*149*) discussed the "shared streets" concept, which provides an area that is shared by pedestrians and vehicles.

It should be noted that the U.S. Access Board has received numerous complaints that speed humps and other vertical deflection measures have a detrimental effect on the health of some people with disabilities. The "Building a True Community" report (*35*) by PROWAAC includes discussions on traffic calming and other issues, and is available online at www.access-board.gov.

Fambro et al. (*151*) developed geometric design guidelines for suburban, high-speed curb and gutter roadways. The resultant guidelines were based on the input from a panel of experts and the results of several safety, operational, and computer simulation studies. They were prepared in a format that could be inserted into a Texas design manual.

Several research projects have investigated the relationships between accidents and cross-section elements. In 1992, the FHWA published the report, *Safety Effectiveness of Highway Design Features, Volume III: Cross Sections* (*153*) that provided a summary of identified relationships between cross-sectional elements and accident experience. Additional information is included in a paper by Zegeer et al. (*154*) that discussed the effects of various roadside features on accident experiences and a paper by Zegeer et al. (*155*) that quantified the benefits expected from lane widening, shoulder widening, shoulder surfacing, and general roadside improvements.

A study (*160*) using data from the Highway Safety Information System examined the effects of various cross-section-related design elements on accident frequency and developed an accident prediction model for rural, multilane, nonfreeway highways. Council and Stewart (*163*) attempted to estimate the benefits of converting a two-lane rural road to a four-lane undivided or divided facility. Predicted crash reductions for conversion from most typical two-lane to four-lane divided sections ranged from 40 to 60 percent. The reduction due to conversion to a four-lane undivided configuration is much less well defined, ranging from no effect to perhaps a 20 percent reduction.

In 1994, Zegeer et al. (*158*) quantified the accident effects of lane and shoulder widths on rural roads carrying fewer than 2,000 vehicles per day. Accident rates on paved, low-volume roads are significantly reduced by wider roadway width, improved roadside condition, flatter terrain, and fewer driveways per 1.6 km (1 mi). The presence of a shoulder is associated with significant accident reductions for roads with lane widths of 3.1 m (10 ft) or greater. The results of the accident data analyses were used along with other considerations in the development of recommended changes to the AASHTO guidelines for roadway widths on low-volume roads. Details of those recommended guidelines are contained in *Roadway Widths for Low-Traffic-Volume Roads* (*159*).

Michie and Bronstad (*161*) conducted an in-depth study of accident data and estimates of the frequency of unreported accidents to determine a more realistic view of guardrail performance. They determined that unreported guardrail impacts represent approximately 90 percent of the total impacts. Assuming no injuries or fatalities in the unreported drive-away accidents, only 6 percent of all guardrail impacts involve any injury or fatality. Turner (*157*) outlined the clear roadside concept and provided specific treatments for several of the most prominent obstacles. Opiela et al. (*162*) reported on strategies for improving roadside safety developed by a group of professionals who were assembled to review the problem, identify possible solutions, and define impediments to resolving the problem. They structured the possible solutions to the roadside safety problem in the form of a strategic plan.

In an analysis of continuous shoulder rumble strips (CSRS), the CSRS sites were associated with a reduction of single-vehicle run-off-the-road accidents ranging from 21.1 to 7.3 percent reduction (*164*). The study estimated that one single-vehicle run-off-the-road accident (at an average cost of $62,200) could be prevented every 3 years based on an investment of $217/km to install rolled-in CSRS. They recommended that widespread implementation be considered.

INTERSECTIONS

Intersection design has a large impact on facility capacity and safety. Design tools and treatments intended for both rural and urban areas have been developed and recommended for use. Suggested changes to future editions of design manuals are listed here.

- Consider introducing some of the alternative intersection designs (e.g., roundabouts) (*167–180*).
- Incorporate findings from *NCHRP Report* 375 on median widths at intersections (*166*).
- Incorporate discussion on determining driveway vertical curves (*192,193*).
- Incorporate findings from *NCHRP Report 383* on intersection sight distance (*181*).
- Incorporate findings from *NCHRP Report 420* on corner clearances (*15*).
- Review the newer methods that determine whether a left-turn lane should be used and that calculate the length of the turn lane to identify whether the methods used should change (*15,195–210*).
- Consider incorporating numeric guidelines for offsetting opposing left-turn lanes (*211,212*).
- Add information on triple left-turn lanes (either specific information or reference) (*216,217*).
- Add information on guidelines for use of right-turn lanes (*219–221*).
- Evaluate intersection channelization needs for trucks (*222–224*).
- Reference appropriate reports on crash models (*235,236*) accident mitigation (*7,37*), and guidelines on making the public rights-of-way accessible to all users (*35*).

Several reports discussed alternative intersection configurations including roundabouts (*167*), converting a major conventional cross intersection into two smaller signalized intersections (*172*), median U-turn design (*174,177*); bowtie, paired intersections, jughandle, continuous flow, and continuous Green T (*174,175*); a quadrant roadway intersection (*176*), superstreet design (*174,177*), grade-separated intersections (*178*), flyover (*179*), and Echelon Interchange (*180*). The "Building a True Community" report by the PROWAAC (available online at www.access-board.gov) contains draft requirements for roundabouts (*35*).

The designs of medians at an intersection were reviewed by Harwood et al. (*166*) and reported in *NCHRP*

Report 375. Specific recommendations were developed for potential inclusion in future editions of the *Green Book* and these recommendations have been adopted by the AASHTO Geometric Design Task Force.

Several papers have examined issues associated with intersection sight distance with the largest study being a NCHRP project. Several recommendations were made within *NCHRP Report 383* including using gap acceptance to determine intersection sight distance (*181*). Zeidan and McCoy (*187*), Gattis and Low (*188*), and Gattis (*189*) reviewed sight distance needs at right-turn lanes, intersections at less than 90 degrees, and intersections where the major roadway curves and the minor road extends or projects from the tangent, respectively. Tipaldo (*190*) discussed practical approaches to improving corner sight distance at urban intersections.

Sharp changes in grades on driveways can inhibit the performance of vehicles and create operational and safety concerns. Williams et al. (*191*) and Eck and Kang (*192*) developed geometric design standards for driveway vertical curves and profiles. The PROWAAC report, “Building a True Community” (*35*), also provides guidelines.

Corner clearance is the distance from an intersection to a driveway. Long and Gan (*194*) developed a model to calculate these distances; however, Gluck et al. (*15*) cautioned regarding its use because of limitations within its development. In *NCHRP Report 420*, Gluck et al. provided planning guidelines for corner clearance and noted that there is a wide range of practices, with values ranging from 16 to 300 ft (*15*).

A previous NCHRP synthesis (*195*) provided information on left-turn treatments at intersections and discussed several methods that are available for determining left-turn lane requirements and left-turn lane lengths including the methods currently available in the *Green Book*. McCoy et al. (*211*) and Tarawneh and McCoy (*212*) developed guidelines for offsets for opposing left-turn lanes. Ackeret developed criteria for the geometric design of triple left-turn lanes (*216*). Guidelines for the use of right-turn lanes were developed for uncontrolled approaches to urban intersections (*219*), at unsignalized intersections on rural two-lane highways (*220*), and at unsignalized intersections on rural two- and four-lane highways (*221*).

Truck performance at intersections was evaluated in three papers (*222–224*). The studies commented on the anticipated changes in truck characteristics (for example, the longer and wider trucks permitted by the Surface Transportation Assistance Act of 1982) and the need to develop channelization guidelines based on the amount of offtracking that the trucks will generate.

Dewar (*225*) and Harkey (*226*) reviewed older pedestrian characteristics at intersections. They both commented that the walking speed of most older pedestrians is less than the assumed walking speed of 4 ft/sec (1.3 m/sec), and provided several recommendations to reduce older driver and pedestrian accidents. The PROWAAC report, “Building a True Community” (*35*), recommends that 3.5 ft/sec (1.1 m/sec) be used as the assumed walking speed to better accommodate disabled pedestrians.

Older driver performance was also investigated as part of the FHWA study that produced the *Older Driver Highway Design Handbook* (*7*), and by Garber and Srinivasan (*227,228*) and Naylor and Graham (*229*). The *Older Driver Highway Design Handbook* includes recommendations for at-grade intersections. Garber and Srinivasan (*227,228*) found that older drivers have a higher potential for committing a traffic violation during a turning maneuver, particularly when making left turns.

Several studies reported on findings from evaluations of accidents at intersections. Parsonson (*231*) recommended the use of prefabricated raised median treatments to reduce the number of inappropriate left turns into driveways that are close to intersections. Bonneson and McCoy (*132*) identified measures used by state highway departments to mitigate the impact of partial access control on high-speed rural expressways. Weerasuriya and Pietrzyk (*233*) developed conflict tables for unsignalized three-legged intersections. Kuciemba and Cirillo (*234*) identified the number of accidents by intersection type and accident rates by average daily traffic for T-type and four-way intersection configurations. A recent NCHRP project produced the report, *Accident Mitigation Guide for Congested Rural Two-Lane Highways* (*37*), which includes a review of the effectiveness of several countermeasures used to improve safety at rural intersections. Recent research projects have developed crash prediction models for two-lane rural segments and intersections (*235*) and four-lane by two-lane Stop-controlled intersections (*236*) and two-lane by two-lane signalized intersections (*236*).

INTERCHANGES

Highway interchanges involve a large number of interrelated factors that can greatly affect their design. Reviews of these factors have resulted in recommendations benefiting safety and operations. The following is a list of issues for consideration in future editions of design manuals:

- Incorporate appropriate findings from *NCHRP Report 345* on single-point urban interchanges (*241*).
- Integrate findings from the comparisons of different interchange types (*241–247*).

- The design policy for ramps should consider the anticipated operating speed of the ramp (*250*).
- Insert material on two-lane loop ramps (*254*).
- Determine if changes are needed to discussion on the at-grade intersection portion of an interchange ramp terminal (*258*).
- Reference the AASHTO *Guide for the Design of High-Occupancy Vehicle Facilities* and add comments regarding the use of entrance ramp meters and HOV bypass lanes (*143,144,256*).
- Add material on urban interchanges (*248,249*).
- Add material on pedestrians at expressway ramps (*249,258*).
- Add additional comments regarding human factors-related issues associated with interchanges, including the needs of older drivers (*7,259,260*).
- Add comments regarding accident experience on ramps (*261–264*).

Messer et al. (*241*) reported on current practices in design and traffic operations of existing single-point urban interchanges (SPUIs) and developed guidelines for the design, traffic operational analysis, and cost-effectiveness of SPUIs in *NCHRP Report 345*. Dorothy et al. (*243*) investigated the appropriateness of SPUIs for use in Michigan. Several papers compared the operational performance of different interchange forms, with many comparing the performance of the SPUI or the diamond interchange to other configurations (*244–247*).

Harwood and Mason (*250*) explored issues inherent in the selection of appropriate ramp design speeds and geometrics to avoid interchange operational and safety problems. They stated that design policy for horizontal curves should ensure an adequate margin of safety against both rollover and skidding at the travel speeds actually used by vehicles on a particular horizontal curve. In other words, it is not enough just to select a design speed for a ramp to fit the physical constraints of the site. There is also a need to determine an anticipated operating speed for the ramp. If a substantial percentage of vehicles is expected to travel faster than the design speed, there is a need to change to a higher design speed or to incorporate effective speed-control measures in the design.

Rajappan and Walton (*252*) assessed the operational impact of longer combination vehicles (LCVs) on the geometry of diamond interchanges located along Interstate highways in Texas. The authors concluded that if the LCVs are introduced into the Texas Interstate highway system, the geometry of the existing diamond interchange ramp terminals needs to be improved significantly to prevent damage to pavement edges and other roadside appurtenances by the rear wheels of the LCVs.

Koepke (*253*) reviewed taper and parallel ramp exit and entrance designs and concluded that current design elements are acceptable for today's driving conditions. He identified that previous research has found that the gore area of exit ramps ranks high in the location of freeway accidents and there are some problems with driver gap acceptance occurring on entrance ramps. Both conditions have been attributed to the assumption that drivers do not know how to properly use, or just do not properly use, speed-change lanes.

Walker (*254*) reviewed the operational, safety, and capacity aspects of existing two-lane loop ramps. He concluded that directional or semi-directional ramps are preferred for high-volume, two-lane ramps. Lomax and Fuhs (*256*) reviewed the geometric design standards and practice of states that operate freeway entrance ramp meters and HOV bypass lanes. They also summarized the 1992 AASHTO *Guide for the Design of High-Occupancy Vehicle Facilities*.

Pate and Stover (*248*) investigated three interchange options for densely developed areas—tight urban diamond interchange, SPUI, and the left-hand exit single signal.

Zeidan et al. (*249*) developed guidelines for the design of pedestrian facilities at interchanges. The development of the guidelines was based on literature review, survey of current practice, the observation of pedestrian behaviors at interchanges, review comments from highway designers, and cost-benefit analysis of sidewalks on bridges. Plummer et al. (*258*) identified key design considerations for the at-grade intersection portion of an interchange ramp terminal and made several recommendations for future geometric design policies and research. The report, *Design and Safety of Pedestrian Facilities* (*33*), also commented about pedestrians at expressway ramps, for example, stating that the "hazard to pedestrians at ramp intersections is often difficult to correct . . . . however, the level of hazard at many of these intersections can be lessened through the use of appropriate traffic-control devices (e.g., warning signs) to reduce vehicle speeds and alert motorists and pedestrians. In some instances, pedestrian barriers, modified signal timing (e.g., longer vehicle clearance intervals), or even grade separation (e.g., pedestrian overpasses) in extreme situations may be needed to reduce a serious pedestrian safety problem."

Lunenfeld (*260*) examined the human factors issues associated with interchange design features. He emphasized that "engineers and designers should be aware of and account for all of them and bear in mind that their efforts are first and foremost to aid the driver." Knoblauch et al. (*259*) identified characteristics of older drivers that affect their ability to drive on freeways and recommended research to develop guidelines for countermeasures to accommodate their needs. The *Older Driver Highway Design Handbook* (*7*) provides specific recommendations for highway design elements for interchanges in the following four areas: exit signing and exit ramp gore delineation, acceleration/deceleration lane design features, fixed lighting installation, and traffic control devices for prohibited movements on freeway ramps.

Several papers discussed safety issues associated with interchanges. Twomey et al. (*261,262*) critically reviewed and summarized past safety research on interchanges. Some of their findings conclude that interchange ramps should be designed with flat horizontal curves (except in rural areas), that sharp curves at the end of ramps and sudden changes from straight alignment to sharp curves should be avoided, and that cloverleaf ramps, left-side ramps, and scissor ramps should be avoided where possible. An FHWA research project conducted by Bauer and Harwood (*263*) developed statistical models to define the relationship between traffic accidents and highway geometric design elements and traffic volumes for interchange ramps and speed-change lanes. Most of the variability was explained by ramp annual average daily traffic (AADT). Other variables found to be significant in some models included mainline freeway AADT, area type (rural/urban), ramp type (on/off), ramp configuration, and combined length of ramp and speed-change lane. In California, ramp accidents accounted for 2.8 percent of the accidents on rural highways and 18.4 percent of accidents on urban highways (*264*). Scissor ramps, rest area ramps, and slip ramp configurations were likely to be associated with high accident rates. Accident rates for on-ramps were consistently lower than those for off-ramps.

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APPENDIX A

Survey

The survey was distributed to the 50 states via e-mail, with 26 states providing a response. Table A-1 provides the opening material of the survey and Table A-2 lists the states responding. The responses for each of the questions (1–5) on the survey are shown in Tables A-3 through A-7.

TABLE A-1
OPENING MATERIAL FOR THE SURVEY

National Cooperative Highway Research Program
NCHRP Synthesis Topic 31-01
QUESTIONNAIRE

RECENT GEOMETRIC DESIGN RESEARCH FOR
IMPROVED SAFETY AND OPERATIONS

The American Association of State Highway and Transportation Officials' (AASHTO) document, *A Policy on Geometric Design of Highways and Streets* (commonly referred to as the *Green Book*), presents the national policy for geometric design. The last version of this policy was published in 1994. During the last decade, there has been considerable research on all aspects of geometric design affecting how roadways are designed, how they operate, and ultimately, the safety of these facilities. A limitation to the potential application of this research is the sheer volume of information that has been published since 1990. A synthesis of recent research is being developed to aid in the implementation of the findings from research. The synthesis will critically review and selectively summarize the geometric design research since 1990, particularly research with safety and operational implications. Areas to be addressed include, but are not limited to, design speed, controls and criteria (e.g., definitions, vehicles, users), horizontal alignment, vertical alignment, cross section (i.e., right-of-way), intersections, interchanges, access management, and design consistency. **The purpose of this survey is to identify recent research in these areas, especially research that has been conducted on a state or local level or has not yet been documented in a published report and/or paper.**

TABLE A-2
STATES RESPONDING

Connecticut	Delaware	Florida	Georgia
Idaho	Illinois	Kansas	Louisiana
Maine	Mississippi	Montana	Nebraska
Nevada	New Hampshire	New Jersey	New York
North Carolina	Oklahoma	Pennsylvania	Tennessee
Texas	Virginia	Washington	West Virginia
Wisconsin	Wyoming		

TABLE A-3
RESPONSES TO QUESTION 1

Question 1: What roadway geometric design policy is used by your agency?		
No.	Responses	
6	A	The *Green Book*.
7	B	The *Green Book* plus supplementary materials that present agency-specific policies.
16	C	A stand-alone state design manual that is largely based on the *Green Book*.
0	D	A stand-alone state design manual that contains significant departures from the *Green Book*.
1	E	Other (please describe): A design manual that references the *Green Book* and other materials.

Note: Numbers do not total 26 because some states circled more than one answer.

TABLE A-4
RESPONSES TO QUESTION 2

Question 2: If you use a design manual other than the *Green Book*, have you revised it within the previous ten years? (21 *Yes*/4 *No*). If *yes*, what sections were revised and what reference materials were used in the revision. (*Yes* answers follow.)	
Y	In-house experience (2R), in-house experience (3R).
Y	Revisions were based on the *Green Book* and the *Roadside Design Guide*.
Y	Entire manual is currently being revised based upon most current information from AASHTO.
Y	General update of design directions based on the *Green Book* and low-volume road criteria.
Y	The *Roadway Design* is based on the *Green Book* plus additional NCDOT policies. The *Roadway Design Manual* should not contradict the *Green Book* in any area.
Y	Horizontal clearance—AASHTO *Green Book*, *Roadside Design Guide*. Intersection sight distance—AASHTO *Green Book*. RRR criteria—AASHTO *Green Book*, *TRB Special Report 214*.
Y	Geometric standards—AASHTO *Green Book*.
Y	All sections *Green Book*—AASHTO *Bike Guide*.
Y	*Roadside Design Guide*—Typical cross-section element, clear distances, etc.
Y	Revised entire manual: Primarily procedural and conversion to metric. The *Green Book*, *Roadside Design Guide*, *Highway Capacity Manual*, and KDOT documents are referred to.
Y	Pavement and shoulder widths and other detailed design based on new MoDOT policies and *Green Book* revisions. Most were due to change in criteria to consider functional class in geometric design.
Y	Completely new publication in 1996.
Y	Several revisions since 1992.
Y	Over the last 10 years WisDOT has revised every one of the 27 chapters we now have in our *Facilities Development Manual* (roadway design manual). The changes are based on NCHRP reports, especially *NCHRP Report 350*, changes in state and federal laws (e.g., erosion control, recycling material in embankment fills or in asphaltic pavement, and new products or techniques). Some of the changes have occurred due to field experience or other internal studies that include literature searches (e.g., passing lanes and rural intersection improvements).
Y	We are in the process of rewriting the entire *Design Manual*. In the last 3 years we have revised approximately two-thirds of the chapters in the manual. The revisions are to incorporate changes in procedures, remove confusing language, and to get the whole manual in a consistent writing style. The changes have primarily been to meet changes in state policy within the limits of the *Green Book*; other reference materials used in each chapter are listed in the Reference section. All of our technical manuals can be found on the Internet at: http://www.wsdot.wa.gov/fasc/EngineeringPublications/library.htm.
Y	Updated all material to take into account the change to metric.
Y	State standards for non-NHS highways were developed in 1995. These standards are being upgraded currently, with the guide due out this month.
Y	MDT's *Road Design Manual* was updated in conjunction with metric conversion by a consultant. The manual is based largely on the *Green Book*, but also the *MDT Geometric Design Standards*.
Y	3R Non-Freeway Projects, SR 214, *Designing Safer Roads*, FHWA Tech. Advisory T5040.28.
Y	State standards developed for non-NHS routes. Don't know what, if any, reference materials were used.
Y	Provided an attached table of contents and bibliography from the IDOT *Design and Environment Manual 2000*. A CD-ROM is available from IDOT at 217/785-8971. An English/Metric edition is in progress with Geometric Design Policy revisions.

TABLE A-5
RESPONSES TO QUESTION 3

Question 3: Has your state funded any research projects that examined geometric design issues (e.g., effects of lane width on safety, appropriate superelevation rates for urban areas, etc.) in the previous ten years? (*9 Yes/18 No*). If *yes*, please list the research projects and/or reports generated from the projects (*listed below*).
TTI (Urbanik): Urban Highway Operations Research and Implementation Program, Project 0-1232, 1989–1994. TTI (Fambro): Design Criteria for Suburban High Speed Curb and Gutter Sections, Project 0-1347, 1992–1994. TTI (Wooldridge): Geometric Design Guidelines for At-Grade Intersections Near Railroad-Highway Grade Crossings, Project 0-1845 (1998–2000). TTI (Wooldridge): Geometric Design Guidelines to Accommodate Incident Management Strategies, Project 0-1848 (1998–1999). TECH (Gransberg): Development of an Improved Two-Lane, Two-Way Highway Geometric Section, Project 7-3951 (1997–1998).
Older Drivers (Underway). Superelevation on Step Grades and Sharp Curves (Underway).
#1. Zegeer, C.V. and J.E. Hummer, “Safety Effectiveness Procedures for North Carolina’s 3R Design Guide,” 1992. #2. Hummer, J.E., “Unconventional Design and Operational Strategies for Oversaturated Suburban Arterials,” 1994. #3. Graham, J.R., “Intersection Design Decision/Reaction Time for Older Drivers,” 1995. Note: NCDOT has also contributed to various FHWA national pooled fund studies.
Gene Russell, K-Tran: KSU-97-1, Review of the Effectiveness Location, Design and Safety of Passing Lane in Kansas, Oct. 1999. Bob Stokes, “Review of Safety Effects of Kansas Intersections with Wide (150 ft) Medians.” K-TRAN: KU-92-3, Operational Analysis of Collector-Distributor Systems, Mulinazzi, Shen, and Schmidt, July 1993. K-TRAN: KSU-93-1, “Evaluation of Corridor Right-of-Way Preservation Programs for Kansas,” Stokes, Russell, and Vellanki, May 1994. K-TRAN: KSU-95-5, “Warrants for Right Turn-Lanes at Unsignalized Intersections,” Hasan and Stokes, May 1996. K-TRAN: KSU-96-3, “Speed Zoning Guidelines Using Roadway Characteristics and Area Development,” Stokes et al., August 1998. K-TRAN:1 KSU-98-1, “Guidelines for Design of 3R Projects for Multiple Design Speeds” (in progress). K-TRAN: KSU-98-6, “Analysis of Rural Intersection Accidents Caused by Stop Sign Violation and Failure to Yield Right-of-Way” (in progress). K-TRAN: KSU-99-1, “Identification of Hump Highway/Rail Crossings” (in progress). K-TRAN: KSU-00-4, “Guidelines for Center of Lane and Shoulder Rumble Strips on Two-Lane Rural Highways,” Russell, Stokes, and Rys (in progress). K-TRAN: KSU-00-6, “Performance of Major Modification Rehabilitation Strategies,” Najjar and Hossain (approved, not started). K-TRAN: KSU-00-7, Roundabout Traffic Patterns, Russell and Rys (in progress).
We have current investigation underway to study the safety and design of Type II median barriers. The study is scheduled for completion 8/31/2000.
Pat McCoy (UNL), Guidelines for Free-Right-Turn Lanes at Unsignaled Rural Intersection, 1994.
Dr. Sicking, Midwest Pool Fund, last few years. This research conducts crash testing on various types of roadside barrier systems. On another subject, we have also provided more detailed guidance on the use of superelevation in urban areas that came from the *Green Book*. We are currently conducting a Freeway System Operational Analysis on the entire Milwaukee area freeway. This may involve geometric changes on a system-wide basis.
Milton, J. and F. Mannering, “The Relationship Among Highway Geometrics, Traffic-Related Elements, and Motor-Vehicle Accident Frequencies,” 1998. Shankar, V.N., R.B. Albin, J.C. Milton, and F.L. Mannering, ”Evaluating Median Crossover Likelihoods with Clustered Accident Counts: An Empirical Inquiry Using the Random Effects Negative Binomial Model,” 1998. Shankar, V., J. Milton, and F. Mannering, “Modeling Accident Frequencies as Zero-Altered Probability Process: An Empirical Inquiry,” 1997. Milton, J.C. and F.L. Mannering, “The Relationship Between Highway Geometrics, Traffic-Related Elements, and Motor Vehicle Accidents,” Final Report, 1996. Shankar, V. and F. Mannering, “Modeling the Endogeneity of Lane-Mean Speeds and Lane-Speed Deviations: A Structural Equations Approach,” 1998. Shankar, V., F. Mannering, and W. Barfield, “Effect of Roadway Geometrics and Environmental Factors on Rural Freeway Accident Frequencies,” 1995.
“Calibration 7 Validation of Capacity Analysis Procedures for Left-Turn Lanes in Illinois” (Nagui Rouphail, Phase I–1992, Phase II–1993). “Cost Effective Roadside Safety Policy for Two-Lane Rural Highways” (David Boyce 1989). “Accident Savings from Roadside Improvements on Two-Lane Rural Highways” (Ray Benekohal 1990). “Freeway Construction Zones in Illinois: A Follow-Up Study” (Also TRR 1035 and 1163) (Nagui Rouphail 1990).

TABLE A-6
RESPONSES TO QUESTION 4

Question 4: If your state has funded a research project on geometric design, have the findings been (or will they be) implemented? (*7 Yes/19 No*). If yes, please describe how the findings are implemented (e.g., revised state design policy on superelevation, change material in publication on driveway location, etc.) (*listed below*).
Projects 0-1347 and 7-3591are being implemented on a trial basis.
Note: numbers correspond to items listed in response to question 3 (Table A-5). #1. 23-page supplement added to design guide; this allows designers to do quick cost-effectiveness evaluations of various design improvements on two-lane roads. #2. None of the recommended designs implemented to date; the NCDOT is continuing to search for appropriate project site(s) application. #3. Results of research validated the currently used value of 2.0 sec for perception–reaction time, as appropriate across all groups.
Program will include provision of passing lanes on selected projects as well as guidance.
Results will be implemented based on study findings.
Revised design policy.
The changes we have implemented in the design manual incorporate the passing crash test results for roadside barrier systems.
Revised IDOT design policies for dual left-turn lane warrants, 3R design guidelines, and freeway construction zones.

TABLE A-7
RESPONSES TO QUESTION 5

Question 5: Please list any research efforts that you believe should be considered as part of a future rewrite of the AASHTO *Green Book*.
Superelevation on steep grades and sharp curves. Neuman, Design criteria for very low-volume roads.
I don't know of any previous efforts. I do recommend that the section dealing with superelevation, on the proper runoff distance, run-out distances, maximum relative gradients, equivalent maximum relatives slopes, maximum comfortable speed on horizontal curves. This may or may not require additional research.
Ross J. Walker, "Two-Lane Loop Ramps: Operation and Design Considerations" *Transportation Research Record 1385*, 1993.
Affects of context sensitive design and design standard flexibility.
Shoulder widths adjacent to auxiliary lanes and left lanes on multilane highways.
Access management plan examples.
None other than those currently being considered for the current *Green Book* rewrite.
Safety and operational effects of "context sensitive" design(s).
Simplification of the superelevation charts.
Dr. Dean Sicking (UNL), Any current guardrail findings (approved), ongoing.
Scientex Corp., FHWA-RD-97-135, *Older Driver Highway Design Handbook*, Jan. 1998. NCHRP Project 17-11, FY 1994, Determination of Safe/Cost Effective Roadside Slopes and Associated Clear Distances, ongoing. NCHRP Project 17-14, FY 1996, Improved Guidelines for Median Safety, ongoing. NCHRP Project 22-17, FY 1999, Recommended Guidelines for Curbs and Curb-Barrier Combinations, ongoing. NCHRP Project 22-11, FY 1994, Evaluation of Roadside Features to Accommodate Vans, Minivans, Pickup Trucks and 4-Wheel Drive Vehicles, ongoing.
Dual dimension, the *Green Book*. Provide guidance on suburban transitional roadways (i.e., where posted speed is between 40 and 50 mph). Expand guidance on context sensitive design as it relates to design standards. Expand guidance on median design (i.e., positive separation distance and median barrier).
Research on new truck speed distance curves, the existing seen outdated and conservative.
Determine if/when there are warrants for right-turn lanes.
Determine if decision sight distance for urban condition is longer than for a rural condition and why?
No specific titles in mind; however, studies on "state standards" should be considered. Also, studies that attempt to quantify or balance geometrics, safety, and environmental inputs.
Any research done on design of roundabout or operation. MRI: Safety Effects of Adding Auxillary Lanes @ At-Grade Intersections. Ongoing research on superelevation, sight distance, and gaps.
NCHRP Project 15-22, Safety Consequences of Flexibility in Highway Design, Pending
Staplin, Lococco, and Byington, FHWA-RD-97-135, *Older Driver Design Handbook*, 1998.
Please consider all references listed in Item 3 above. Only 10 to 20 percent of IDOT's Annual Highway Improvement Program is currently covered by the *Green Book*. Please see attached abstracts. IDOT *Design and Environment Manual 2000*, Chapters 35 (Access Management), 36 (Intersections), 37 (Interchanges), and 45 (Expressways) should also be considered in regard to closely spaced intersections and traffic back-ups.

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