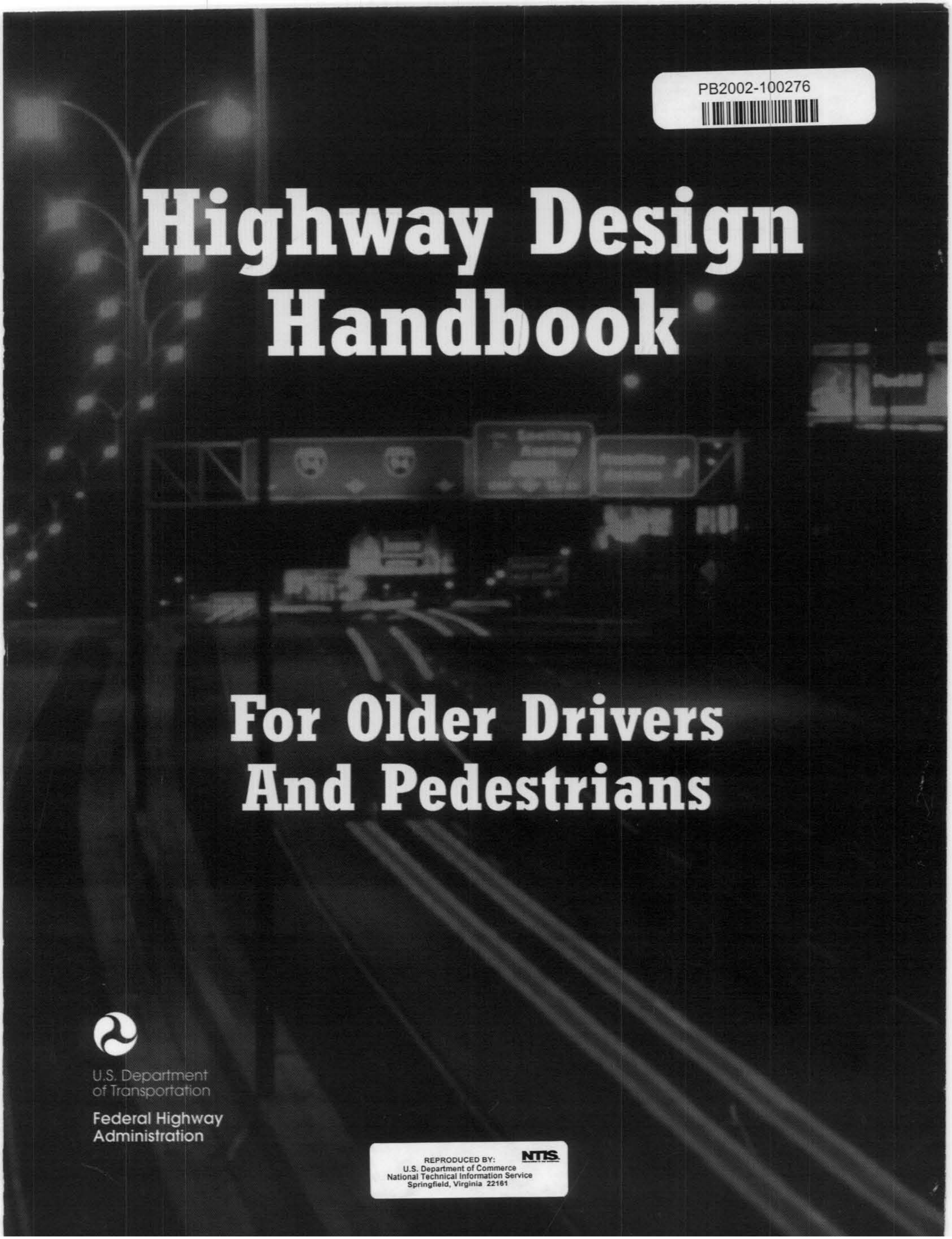

Highway Design Handbook

For Older Drivers And Pedestrians

U.S. Department of Transportation

Federal Highway Administration

FOREWORD

The proportion of the population over age 65 is growing significantly. Older road users can be expected to have problems driving and as pedestrians, given the known changes in their perceptual, cognitive, and psychomotor performances. These changes present many challenges to transportation engineers, who must ensure system safety while increasing operational efficiency.

This *Highway Design Handbook for Older Drivers and Pedestrians* provides practitioners with a practical information source that links older road user characteristics to highway design, operational, and traffic engineering recommendations by addressing specific roadway features. This Handbook supplements existing standards and guidelines in the areas of highway geometry, operations, and traffic control devices.

The information in this Handbook should be of interest to highway designers, traffic engineers, and highway safety specialists involved in the design and operation of highway facilities. In addition, this Handbook will be of interest to researchers concerned with the issues of older road user safety and mobility.

Copies of this report can be obtained through the FHWA Research and Technology Report Center, 9701 Philadelphia Court, Unit Q, Lanham, Maryland 20706, telephone: (301) 577-0818, fax: (301) 577-1421, or the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161, telephone: (703) 487-4650, fax: (703) 321-8547.

Michael F. Trentacoste
Director, Office of Safety
Research and Development

NOTICE

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Contracting Officer's Technical Representatives (COTRs):
Joseph Moyer, HRDS-07; Elizabeth Alicandri, HSA-20

16. Abstract

This project updated, revised, and expanded the scope of the *Older Driver Highway Design Handbook* published by FHWA in 1998. The resulting document incorporates new research findings and technical developments; extensive feedback from State, county, and municipal engineers who reviewed and applied recommendations from the earlier version of the Handbook; and recommendations with supporting background material for aspects of modern roundabouts and highway-rail grade crossings, two elements that were not covered in the 1998 publication. Recommendations geared to use of highway facilities by pedestrians also receive greater emphasis. Guidance on how and when to implement the included recommendations has been added, as well as codes that indicate at a glance the relationship of each recommendation with standard design manuals, including the *MUTCD* and the AASHTO *Green Book*.

Development of the updated Handbook was complemented in this project by a technology transfer initiative to make practitioners aware of the Handbook and assist in applying its recommendations. This included the development of a condensed document, *Guidelines and Recommendations to Accommodate Older Drivers and Pedestrians* (FHWA-RD-01-051), plus printed and electronic materials supporting the conduct of practitioner workshops throughout the United States in the 1999-2001 period.

All research products developed under this contract are designed to provide practical guidance to engineers to accommodate the needs and functional limitations of an aging population of road users. Included recommendations are directed toward new construction, reconstruction, scheduled maintenance, and "spot treatments" to ameliorate demonstrated safety problems, emphasizing countermeasures with modest additional cost during installation and the potential for a cost-savings over the life cycle.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

ACKNOWLEDGMENTS

The quality and usefulness of the *Highway Design Handbook for Older Drivers and Pedestrians* is a direct result of the many highway engineering practitioners and researchers who provided their comments and criticisms to the authors of this document. Beginning with responses to a detailed, two-part survey conducted early in the Handbook development, 94 practitioners contacted through 5 national committees identified the most important content for this resource and how it should be organized and presented for maximum accessibility by its intended users. Participating committees in this user requirements analysis included the American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Design; the National Committee on Uniform Traffic Control Devices (NCUTCD); the AASHTO Standing Committee on Highway Traffic Safety; the Transportation Research Board (TRB) Committee on Geometric Design (A2A02); and the TRB Committee on the Operational Effects of Geometrics (A3A08).

Following development of a draft document, a review panel composed of practicing engineers was asked to apply draft recommendations for one or more design elements from the Handbook in case studies involving real-world engineering problems, and to identify needed changes in the information presented in the Handbook. This panel was formed through the active support of three key committee chairmen: Thomas Warne, AASHTO Subcommittee on Design; Richard Weaver, AASHTO Subcommittee on Traffic Engineering; and Ken Kobetsky, National Committee on Uniform Traffic Control Devices. The 28 State and local engineers who served on this panel, giving freely of their time and talents, made invaluable contributions to the resulting product-the *Older Driver Highway Design Handbook* published by FHWA in 1998. They are acknowledged individually in that document.

Concurrent with the update of the 1998 Handbook, a series of practitioner workshops were held throughout the United States to increase awareness of this resource and to help educate engineers about the functional limitations of older drivers and pedestrians, and how to accommodate them through design and operational enhancements. These workshops, which provided contact with more than 500 State DOT staff in a 3-year period, elicited verbal and written feedback from participants regarding Handbook deficiencies and needed improvements. While it is not feasible to name these participants, it is important to acknowledge their comments as the most powerful influence in shaping changes to specific recommendations and in defining the need for, and manner of, cross-referencing current Handbook recommendations against standard design manuals.

By integrating the feedback from workshop participants with the synthesis of recent research, a draft version of the new Handbook was developed. Additional reviews of this draft followed in 2000, involving a diverse group of FHWA and State-level experts. Headquarters staff as well as staff in FHWA Division offices examined the present recommendations from the standpoint of their merit and also their compatibility with the *MUTCD 2000*; the State-level experts added a critical perspective on the feasibility and best strategies for implementation of *Handbook* recommendations.

State-level experts making key contributions to the updated Handbook are listed below:

N. Alexander
New York State Department of Transportation

Peter Amakobe
Wisconsin Department of Transportation

Rick Beck
Minnesota Department of Transportation

Jon Bray
New York State Department of Transportation

Jim Bryden
New York State Department of Transportation

Paul J. Carlson
Texas Transportation Institute

James M. Ercolano
New York State Department of Transportation

Charles Guenzel
New Jersey Department of Transportation

Zubair Haider
Indiana Department of Transportation

Carol A. Hennessy
New York State Department of Transportation

Carla W. Holmes
Georgia Department of Transportation

Ron King
North Carolina Department of Transportation

Sue Lodahl
Minnesota Department of Transportation

Patrick McCoy
University of Nebraska-Lincoln

Gary Metzer
Wisconsin Department of Transportation

Marchon Miller
Nevada Regional Transportation Commission

G.G. (Buddy) Murr
North Carolina Department of Transportation

Barbara O'Rourke
New York State Department of Transportation

Charles Riedel
New York State Department of Transportation

Gary Ries
Minnesota Department of Transportation

Robert Swanson
Minnesota Department of Transportation

Linda Voss
Kansas Department of Transportation

Micheal Weiss
Minnesota Department of Transportation

Mitch Wiebe
Minnesota Department of Transportation

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Loren Staplin, Kathy H. Lococo, Stanley R. Byington, David L. Harkey
May 2001

PREFACE

The increasing numbers and percentages of older drivers using the Nation's highways in the decades ahead will pose many challenges to transportation engineers, who must ensure system safety while increasing operational efficiency. The 65 and older age group, which numbered 33.5 million in the United States in 1995, will grow to more than 36 million by 2005 and will exceed 50 million by 2020, accounting for roughly one-fifth of the population of driving age in this country. In effect, if design is controlled by even 85th percentile performance requirements, the "design driver" of the early 21st century will be an individual over the age of 65.

There are important consequences of the changing demographics in our driving population. Traffic volumes will increase, problems with congestion will become more widespread, and the demands on drivers will grow significantly beyond present-day operating conditions. At the same time, maintenance of one's health and personal dignity and, in fact, the overall quality of life for older persons depends to an extraordinary degree on remaining independent. Independence requires mobility and, in our society, the overwhelming choice of mobility options is the personal automobile. This means that there will be a steadily increasing proportion of drivers who experience declining vision; slowed decision-making and reaction times; exaggerated difficulty when dividing attention between traffic conflicts and other important sources of motorist information; and reductions in strength, flexibility, and general fitness.

In 1998, FHWA published the *Older Driver Highway Design Handbook,* seeking to provide highway designers and engineers with a practical information source linking the declining functional capabilities of these highway users to design, operational, and traffic engineering enhancements keyed to specific roadway features. Experience with the included recommendations, including extensive feedback from local- and State-level practitioners, indicated a need to revise and update this resource. The result is the *Highway Design Handbook for Older Drivers and Pedestrians*. Recent research has been incorporated, format and content changes have been made to improve its usefulness, guidance on how to implement its recommendations has been added, and the range of applications covered by the Handbook has expanded relative to the 1998 document.

The main body of the Handbook is organized according to five broad site types, each containing one or more specific roadway features with associated design elements. The top priority is *at-grade intersections*, reflecting older drivers' most serious crash problem area. Next, older driver difficulties with merging/weaving and lane-changing operations focus attention on *interchanges (grade separation)*. *Roadway curvature and passing zones,* plus highway *construction/work zones,* are included for two reasons: (1) heightened tracking (steering) demands may increase the driver's workload, and (2) there is an increased potential for unexpected events requiring a swift driver response. Finally, *highway-rail grade crossings* are identified as sites where conflicts are rare (and thus unexpected) and where problems of detection (with passive controls) are exaggerated due to sensory losses with advancing age.

Recommendations for all design elements covered in the Handbook are presented initially, followed by a more lengthy section presenting the Rationale and Supporting Evidence for each recommendation. Within each of these two major Handbook sections, material is organized in terms of five subsections, corresponding to the broad site types noted above. Preceding the recommendations, a section titled "How To Use This Handbook" explains codes used throughout the document to cross-reference the *MUTCD*, *Green Book,* and other standard manuals, and suggests a structured approach to reaching decisions about when to implement Handbook recommendations. The Handbook concludes with an integrated glossary providing definitions of selected terms; a reference list; and an index to help locate Handbook entries pertaining to a particular design element.

The recommendations in this Handbook are based on supporting evidence drawn from a comprehensive review of research addressing human factors and highway safety. The results of field studies employing older drivers were always given precedence, followed by laboratory simulations or modeling efforts where both age and some aspect of highway design, operations, or traffic control were included as study variables. More general findings on the effects of aging, independent of driver performance research *per se*, may also be cited, but only where there is a clear logic extending a given finding to the highway context. A broader discussion of issues related to aging and driving can be found in the Transportation Research Board's *Special Report 218* (1988) and in a pending TRB publication, *A Decade of Experience*, that updates the 1988 volume.

It is essential to recognize that the Handbook recommendations, as well as the evidence cited to support them, relate to demonstrated performance deficits of *normally aging* drivers and pedestrians. Thus, diminished driver capabilities that result from the onset of Alzheimer's disease and related dementias, which are believed to afflict more than 10 percent of those age 65 and older and nearly 50 percent of those age 85 and older, are not the current focus.

To close, it should be emphasized that the recommendations presented in this Handbook do not constitute a new standard of required practice for the included highway design elements. The final decision about when and where to apply each Handbook recommendation remains at the discretion of the practitioner. Hopefully, this resource can be applied preemptively to enhance safety wherever there are large numbers of older drivers in the traffic stream in a given jurisdiction; or, some may employ it primarily as a "problem solver" at older driver crash sites. As a practical matter, it is recognized that the application of Handbook recommendations may be limited to the design of new facilities and to planned highway reconstruction projects. Furthermore, the recommendations contained herein seek to avoid "optimum" solutions that may be unattainable using current materials or practices or that will result in situations where extreme costs are incurred for small anticipated gains in system safety. Ultimately, the contents of this Handbook are intended to provide guidance that-based on the current state-of-the-knowledge of the special needs of normally aging seniors-can be expected to 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.

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ABBREVIATIONS AND ACRONYMS

AAAFTS . . . . American Automobile Association Foundation for Traffic Safety

AADT . . . . annual average daily traffic

AASHTO . . . . American Association of State Highway and Transportation Officials

ASTM . . . . American Society for Testing and Materials

ATSSA . . . . American Traffic Safety Services Association

cd . . . . candela

CIE . . . . Commission Internationale de l'Eclairage

CIL . . . . complete interchange lighting

CMS . . . . changeable message sign

CSSB . . . . concrete safety-shaped barrier

DS . . . . diverge steering

DSD . . . . decision sight distance

FARS . . . . Fatal Accident Reporting System

FHWA . . . . Federal Highway Administration

fL . . . . footlambert

GSA . . . . gap search and acceptance

hfc . . . . horizontal footcandle

IA . . . . initial acceleration

IIHS . . . . Insurance Institute for Highway Safety

ISBL . . . . in-service brightness level

ISD . . . . intersection sight distance

ISTEA . . . . Intermodal Surface Transportation Efficiency Act

ITE . . . . Institute of Transportation Engineers

LI . . . . legibility index

LPI . . . . leading pedestrian interval

MOE . . . . measure of effectiveness

MRVD . . . . minimum required visibility distance

MSC . . . . merge steering control

MUTCD . . . . *Manual on Uniform Traffic Control Devices for Streets and Highways*

ABBREVIATIONS AND ACRONYMS
(continued)

NCHRP . . . . National Cooperative Highway Research Program

NHTSA . . . . National Highway Traffic Safety Administration

NTOR . . . . no turn on red

NTSB . . . . National Transportation Safety Board

PIL . . . . partial interchange lighting

PMD . . . . post-mounted delineator

PRT . . . . perception-reaction time

RPM . . . . raised pavement markers

RT . . . . reaction time

RTOR . . . . right-turn-on-red

SC . . . . steering control

SCL . . . . speed-change lane

SSD . . . . stopping sight distance

STV . . . . small target visibility

SU . . . . single unit (truck)

TCD . . . . traffic control device

TRB . . . . Transportation Research Board

TVA . . . . transient visual adaptation

TWLTL . . . . two-way, left-turn lane

VC . . . . visual clear

HOW TO USE THIS HANDBOOK

RELATING RECOMMENDATIONS TO STANDARD DESIGN GUIDES

Codes placed outside and to the left of each recommendation in this Handbook indicate its relationship to the design guides most frequently referenced by practitioners, as determined by the Handbook authors. An example is shown below.

> **Recommendations by Design Element**
>
> *A. Design Element: Intersecting Angle (Skew)*
>
> AASHTO:1
ICG:1
ITE:1
>
> (1) In the design of new facilities or redesign of existing facilities where right-of-way is not restricted, all intersecting roadways should meet at a 90-degree angle.

Relationship codes 1 through 4, plus a fifth code (IEC), are defined as follows:

1 Handbook recommendation selects the most conservative design value among present options in the standard manual/guideline. (*Example*: Using a larger sign size identified as an "option" in the *MUTCD*.)

2 Handbook recommendation indicates the preferred design value where a discrepancy exists between current standards/guidelines. (*Example*: Limit skew to 75 degrees as per ITE instead of 60 degrees as per AASHTO.)

3 Handbook recommendation extends a current practice to a new application or operation. (*Example*: Use of fluorescent sheeting on wrong-way control signing for increased conspicuity.)

4 Handbook recommendation advances a specific design value where only general guidance now exists, or provides more detailed or more stringent design criteria than are currently specified. (*Example*: Assume 0.4 m of visibility per mm [33 ft per inch] of letter height on highway signing, not 0.6 m/mm [50 ft/in] as in *MUTCD* 1988, or even 0.5 m/mm [40 ft/in] as proposed for *MUTCD* 2000.)

IEC Handbook recommendation is permissible at this time only in accordance with the provisions of *MUTCD* section 1A.10, Interpretations, Experimentations, and Changes.

These recommendations represent advances in technology that research indicates will result in improved safety and efficiency of operations.

The standard design guides referenced by the relationship codes in the example above and throughout the Handbook are listed below. The most current published edition of each guide was consulted in the preparation of the Handbook, with the exceptions as noted.

AASHTO	*A Policy on Geometric Design of Highways and Streets [AASHTO Green Book].* American Association of State Highway and Transportation Officials, 1994.
HCM	*Highway Capacity Manual.* Transportation Research Board, 1999. (Special Report 209)
ICG	*Intersection Channelization Design Guide.* National Cooperative Highway Research Program, 1985. (Report No. 279)
ITE	*Traffic Engineering Handbook.* Institute of Transportation Engineers, 1999.
MUTCD	*Manual on Uniform Traffic Control Devices for Streets and Highways.* Federal Highway Administration, 2000.
RLH	*Roadway Lighting Handbook.* Federal Highway Administration, 1978. Implementation Package 78-15. (Reprinted April 1984) [NOTE: Although an Addendum to chapter 6 of the *Roadway Lighting Handbook* was produced in 1983, the recommendations pertaining to the *RLH* primarily reference material found in the chapters produced in the 1978 version.]
RND	*Roundabouts: An Informational Guide.* Federal Highway Administration, 2000.
RRX	*Railroad-Highway Grade Crossing Handbook.* Federal Highway Administration, 1986.

INTERPRETING HANDBOOK GRAPHICS

The included figures and drawings are for illustrative purposes only, to clarify the meaning of a recommendation or to show what design was employed in a research study referenced in the Rationale and Supporting Evidence section. It is important to note that the fonts and arrow graphics used in this Handbook are not always consistent with the *MUTCD*-approved fonts and arrows. When employing recommendations included in this Handbook, only *MUTCD*-approved fonts and arrow graphics should be used.

USING THE TIME-SPEED-DISTANCE TABLE

A number of recommendations presented in the Handbook identify the placement of a device or treatment in terms of the preview time that should be provided to the driver for its application. These values are typically expressed in seconds, such that the recommended placement of the device or treatment depends upon the speed at which traffic is moving. To facilitate application of Handbook recommendations of this nature, a look-up table on the next page provides the advance placement distance needed to achieve a desired preview time at a particular operating speed.

Advance Placement Distances Required to Achieve Desired Preview Times at Designated Operating Speeds

Preview Time (seconds)	Operating Speed										
	48 km/h (30 mi/h)	56 km/h (35 mi/h)	64 km/h (40 mi/h)	72 km/h (45 mi/h)	80 km/h (50 mi/h)	88 km/h (55 mi/h)	97 km/h (60 mi/h)	105 km/h (65 mi/h)	113 km/h (70 mi/h)	121 km/h (75 mi/h)	129 km/h (80 mi/h)
2.5	34 m (110 ft)	39 m (128 ft)	45 m (147 ft)	50 m (165 ft)	56 m (183 ft)	62 m (202 ft)	67 m (220 ft)	73 m (238 ft)	78 m (257 ft)	84 m (275 ft)	89 m (293 ft)
3.0	40 m (132 ft)	47 m (154 ft)	54 m (176 ft)	60 m (198 ft)	67 m (220 ft)	74 m (242 ft)	81 m (264 ft)	87 m (286 ft)	94 m (308 ft)	101 m (330 ft)	107 m (352 ft)
3.5	47 m (154 ft)	55 m (180 ft)	63 m (205 ft)	70 m (231 ft)	78 m (257 ft)	86 m (282 ft)	94 m (308 ft)	102 m (334 ft)	110 m (359 ft)	117 m (385 ft)	125 m (411 ft)
4.0	54 m (176 ft)	63 m (205 ft)	72 m (235 ft)	81 m (264 ft)	89 m (293 ft)	98 m (323 ft)	107 m (352 ft)	116 m (381 ft)	125 m (411 ft)	134 m (440 ft)	143 m (469 ft)
4.5	60 m (198 ft)	70 m (231 ft)	81 m (264 ft)	91 m (297 ft)	101 m (330 ft)	111 m (363 ft)	121 m (396 ft)	131 m (429 ft)	141 m (462 ft)	151 m (495 ft)	161 m (528 ft)
5.0	67 m (220 ft)	78 m (257 ft)	89 m (293 ft)	101 m (330 ft)	112 m (367 ft)	123 m (403 ft)	134 m (440 ft)	145 m (477 ft)	157 m (513 ft)	168 m (550 ft)	179 m (587 ft)
5.5	74 m (242 ft)	86 m (282 ft)	98 m (323 ft)	111 m (363 ft)	123 m (403 ft)	135 m (444 ft)	148 m (484 ft)	160 m (524 ft)	172 m (565 ft)	185 m (605 ft)	197 m (645 ft)
6.0	81 m (264 ft)	94 m (308 ft)	107 m (352 ft)	121 m (396 ft)	134 m (440 ft)	148 m (484 ft)	161 m (528 ft)	174 m (572 ft)	188 m (616 ft)	201 m (660 ft)	215 m (704 ft)
6.5	87 m (286 ft)	102 m (334 ft)	116 m (381 ft)	131 m (429 ft)	145 m (477 ft)	160 m (524 ft)	174 m (572 ft)	189 m (620 ft)	204 m (667 ft)	218 m (715 ft)	233 m (763 ft)
7.0	94 m (308 ft)	110 m (359 ft)	125 m (411 ft)	141 m (462 ft)	157 m (513 ft)	172 m (565 ft)	188 m (616 ft)	204 m (667 ft)	219 m (719 ft)	235 m (770 ft)	250 m (822 ft)
7.5	101 m (330 ft)	117 m (385 ft)	134 m (440 ft)	151 m (495 ft)	168 m (550 ft)	185 m (605 ft)	201 m (660 ft)	218 m (715 ft)	235 m (770 ft)	252 m (825 ft)	268 m (880 ft)
8.0	107 m (352 ft)	125 m (411 ft)	143 m (469 ft)	161 m (528 ft)	179 m (587 ft)	197 m (645 ft)	215 m (704 ft)	233 m (763 ft)	250 m (822 ft)	268 m (880 ft)	286 m (939 ft)

KNOWING WHEN TO IMPLEMENT THESE RECOMMENDATIONS

Implementation of the recommendations in this Handbook is expected to provide remedies for design deficiencies that disproportionately penalize older road users due to changes in functional ability experienced with normal aging. These may be most urgently needed where a crash problem with older drivers or pedestrians has already been demonstrated; however, the greater benefit arguably lies in designing safer new roads and identifying and modifying problems with existing roads before statistics reveal a crash problem. Not only does this practice minimize the risk and severity of crashes, it minimizes the need for remedial work after construction, thus reducing the whole-life cost of projects. This is the central premise of the *road safety audit* process supported by FHWA (1997) and ITE (1995), and it holds the key for applying the Handbook's recommendations as well.

The engineering enhancements described in this document should benefit all road users, not just older persons. However, if higher construction costs, the need for additional right-of-way, or other factors are present, special justification may be required for implementation of Handbook practices. This section was developed to support engineering judgment in this regard. It suggests a three-step procedure using checklist responses plus brief written comments, as explained below. A separate Implementation Worksheet for meeting the requirements of each step is also provided. It is assumed that DOTs have in place processes that define when a crash pattern or a safety problem is evident; this Handbook does not address this level of analysis. Furthermore, it is recognized that States may already follow processes that make the approach described in this section unnecessary. FHWA has no desire to interfere with any procedures used by States that take the same information into account and accomplish the same ends as the three-step procedure below:

Step 1: Problem Identification *[see worksheet on page 6]*

During the planning stage for each project involving new construction or reconstruction of an existing facility, practitioners are asked to determine whether a problem with the safe use of the facility by older drivers and pedestrians currently exists *or* may reasonably be expected based on current and projected use patterns. Using the first worksheet that follows this discussion, problem identification can be accomplished by checking YES or NO to the following four questions:

Q1. "Is there a demonstrated crash problem with older drivers or pedestrians?"

Q2. "Has any aspect of design or operations at the project location been associated with complaints to local-, municipal-, or county-level officials from older road users *or* are you aware of a potential safety problem at this location, either through personal observation or agency documentation, applying your own engineering judgment?"

Q3. "Is this project located on a direct link to a travel origin or destination for which, in the judgment of local planning/zoning authorities or other local officials, older persons constitute a significant proportion of current users?"

Q4. "Is the project located in a census tract or zip code designation that has experienced an increase in the proportion of (non-institutionalized) residents age 65 and older, for the most recent period in which the population was sampled?"

To answer these questions, practitioners will need to obtain reliable crash data from the appropriate division or bureau of their departments of transportation. At least the three most recent years for which data are available should be examined, and the data should be sorted by age, at a minimum. Sources of information outside of the State DOT also may be required to answer the problem identification questions. Potential sources include, but are not limited to:

- Local government officials/board of supervisors/city council representatives.
- Local and State police.
- The (State) Department of Aging and/or county area Agency on Aging.
- The (State) Department of Health and Human Services and Department of Public Welfare.
- The regional planning commission.

Step 2: Identification of Candidate Handbook Applications *[see worksheet on page 7]*

For each project where a practitioner has answered YES to one or more of the problem identification questions in Step 1, the next step is to identify every design element at the to-be-constructed facility for which a recommendation is included in the Handbook. These recommendations should be listed. Then, for each one, the engineer should indicate whether the recommended practice differs from standard State or local practices, and if yes, what additional benefits are expected to result from implementing the applicable Handbook recommendation(s). One possible example of how such worksheet entries could be made is shown below.

Design Elements Addressed by Handbook Recommendations	Applicable Handbook Recomm.	Differs From Existing State or Local Practice?		If YES...	
		NO	YES	Explain Difference	Identify Expected Benefits
IA. Intersection Angle (Skew)	IA(3)		✔	According to *MUTCD* warrants, there is "adequate" sight distance and fewer than 3 RTOR crashes annually on approach.	Should reduce the difficulty for older drivers to check for approaching traffic, and also reduce aggressive behavior of following drivers who don't accept an older driver's decision not to turn on red.
IJ. Street-Name Signing	IJ(1)	✔			

Step 3: Implementation Decision *[see worksheet on page 8]*

To begin Step 3, each Handbook recommendation identified as a candidate for implementation in Step 2 should be properly referenced [e.g., I.E.4(4a)]. Next, any factors related to increased costs (for an enhanced treatment), added approvals that may be needed, or any other special considerations that impact implementation may be noted in separate columns on the worksheet. The final step is then to proceed to an implementation decision. This is recorded as a judgment by the engineer as to whether implementation of the candidate countermeasure is recommended. The engineer's judgment is indicated by a check in the space next to YES or NO in the last column on the worksheet, accompanied by his/her initials for verification. Additional comments should be entered as deemed appropriate.

Step 1: Problem Identification/Project Review Worksheet for *Highway Design Handbook for Older Drivers and Pedestrians*

Project Title/ID: ______________________________

Person Completing Worksheet: ____________________ Date: __________

Q1. "Is there a demonstrated crash problem with older drivers or pedestrians?"

NO ____

YES ____

Source(s): Date of Contact:

______________________________ __________

______________________________ __________

Q2. "Has any aspect of design or operations at the project location been associated with complaints to local-, municipal-, or county-level officials from older road users *or* are you aware of a potential safety problem at this location, either through personal observation or agency documentation, applying your own engineering judgment?"

NO ____

YES ____

Source(s): Date of Contact:

______________________________ __________

______________________________ __________

Q3. "Is this project located on a direct link to a travel origin or destination for which, in the judgment of local planning/zoning authorities or other local officials, older persons constitute a significant proportion of current users?"

NO ____

YES ____

Source(s): Date of Contact:

______________________________ __________

______________________________ __________

Q4. "Is the project located in a census tract or zip code designation that has experienced an increase in the proportion of (non-institutionalized) residents age 65 and older, for the most recent period in which the population was sampled?"

NO ____

YES ____

Source(s): Date of Contact:

______________________________ __________

______________________________ __________

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Implementation Worksheet for *Highway Design Handbook for Older Drivers and Pedestrians*

Project Title/ID: ____

Person Completing Worksheet: ______________________________ Date: ___________

Step 2: Identification of Candidate Handbook Applications

Identify design elements for which a recommendation exists in the Handbook and the applicable recommendations. Then: (a) describe differences between the recommendation and standard practice, and (b) list benefits expected to result from implementing the Handbook.

Design Elements Addressed by Handbook Recommendations	Applicable Handbook Recomm.	Differs From Existing State or Local Practice?		If YES...	
		NO	YES	Explain Difference	Identify Expected Benefits

Implementation Worksheet for *Highway Design Handbook for Older Drivers and Pedestrians*

Project Title/ID: ______________________________

Person Completing Worksheet: ______________________________ Date: ______________

Step 3: Implementation Decision

List each recommendation identified as being a candidate for implementation. Document whether additional approval is needed, and whether increased costs or other special considerations may impact implementation. Based on these considerations, then decide whether or not implementation can be recommended. Check YES or NO, and add your initials to record your judgment. Add supplemental comments as deemed appropriate.

Candidate Handbook Recommendation	Implementation Considerations			Implementation Recommended?
	Added Costs?	Added Approvals?	Other	
				NO _____ Initials _____ YES _____ Comments:
				NO _____ Initials _____ YES _____ Comments:

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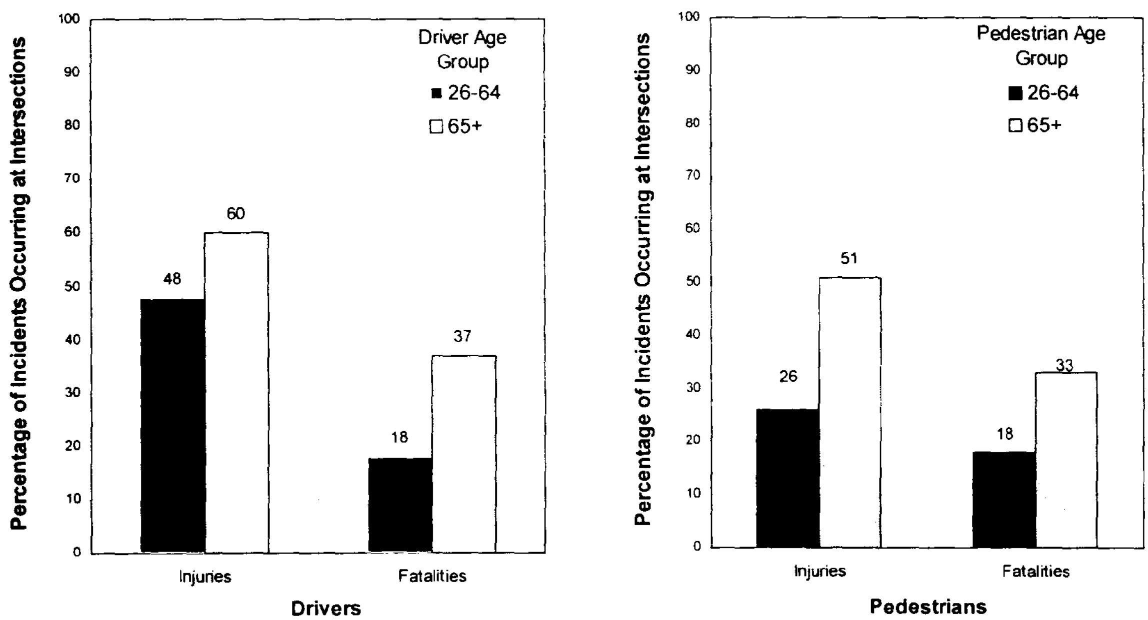
RECOMMENDATIONS

I. INTERSECTIONS (AT-GRADE)

Background and Scope of Handbook Recommendations

The single greatest concern in accommodating older road users, both drivers and pedestrians, is the ability of these persons to negotiate intersections safely. The findings of one widely cited analysis of nationwide crash data (Hauer, 1988), illustrated below, reveal an enduring relationship between injuries and fatalities at intersections in the United States as a function of age and road user type (driver or pedestrian).



For drivers 80 years of age and older, *about half* of fatal crashes occur at intersections (48 to 55 percent), compared with 23 percent or less for drivers up to 50 years of age (FARS 1998 data, in IIHS, 2000). Thirty-eight percent of pedestrian deaths among people age 65 and older in 1998 occurred at intersections (IIHS, 2000). These findings reinforce a long-standing recognition that driving situations involving complex speed-distance judgments under time constraints-the typical scenario for intersection operations-are more problematic for older drivers and pedestrians than for their younger counterparts. Other studies within the large body of evidence showing dramatic increases in intersection crash involvements as driver age increases have associated specific crash types and vehicle movements with particular age groups, linked in some cases to the driving task demands for a given maneuver (Campbell, 1993; Council and Zegeer, 1992; Staplin and Lyles, 1991).

Another approach to characterizing older driver problems at intersections was employed by Brainin (1980), who used in-car observations of driving behavior with 17 drivers ages 25–44, 81 drivers ages 60–69, and 18 drivers age 70 and older, on a standardized test route. The two older age groups showed more difficulty making right and left turns at intersections and responding to traffic signals. The left-turn problems resulted from a lack of sufficient caution and poor positioning on the road during the turn. Right-turn difficulties were primarily a result of failing to signal. Older drivers also displayed difficulty during their approach to an intersection. Errors demonstrated at stop signs included failing to make complete stops, poor vehicle positioning at stop signs, and jerky and abrupt stops. Errors demonstrated at traffic signals included stops that were either jerky and abrupt, failure to stop when required, and failure to show sufficient caution during the intersection approach.

Complementing crash analyses and observational studies with subjective reports of intersection driving difficulties, a statewide survey of 664 senior drivers by Benekohal, Resende, Shim, Michaels, and Weeks (1992) found that the following activities become more difficult for drivers as they grow older (with proportion of drivers responding in parentheses):

- Reading street signs in town (27 percent).
- Driving across an intersection (21 percent).
- Finding the beginning of a left-turn lane at an intersection (20 percent).
- Making a left turn at an intersection (19 percent).
- Following pavement markings (17 percent).
- Responding to traffic signals (12 percent).

Benekohal et al. (1992) also found that the following highway features become more important to drivers as they age (with proportion of drivers responding in parentheses):

- Lighting at intersections (62 percent).
- Pavement markings at intersections (57 percent).
- Number of left-turn lanes at an intersection (55 percent).
- Width of travel lanes (51 percent).
- Concrete lane guides (raised channelization) for turns at intersections (47 percent).
- Size of traffic signals at intersections (42 percent).

Comparisons of responses from drivers ages 66–68 versus those age 77 and older showed that the older group had more difficulty following pavement markings, finding the beginning of the left-turn lane, and driving across intersections. Similarly, the level of difficulty for reading street signs and making left turns at intersections increased with increasing senior driver age. Turning left at intersections was perceived as a complex driving task. This was made more difficult when raised channelization providing visual cues was absent and only pavement markings designated which were through lanes versus turning lanes ahead. For the oldest age group, pavement markings at intersections were the most important item, followed by the number of left-turn lanes, concrete guides, and intersection lighting. A study of older road users completed in 1996 provides evidence that the single most challenging aspect of intersection negotiation for this group is performing left turns during the permitted (steady circular green indication) signal phase (Staplin, Harkey, Lococo, and Tarawneh, 1997).

During focus group discussions (Benekohal et al., 1992), older drivers reported that intersections with too many islands are confusing; raised curbs that are unpainted (unmarked) are difficult to see; and textured pavements (rumble strips) are of value as a warning of upcoming raised medians, approaches to (hidden or flashing red) signals, and the roadway edge/shoulder lane boundary. Study subjects indicated a clear preference for turning left on a protected arrow phase, rather than making permitted-phase turns. When turning during a permitted phase of signal operation, they reported waiting for a large gap before making a turn, which frustrates drivers in back of them. A key finding was the need for more time to react.

Additional insight into the problems older drivers experience at intersections was provided by focus group responses from 81 older drivers (Staplin et al., 1997). The most commonly reported problems are listed below:

- Difficulty in turning their heads at skewed (non-90-degree) angles to view intersecting traffic.
- Difficulty in smoothly performing turning movements at tight corners.
- Hitting raised concrete barriers such as channelizing islands in the rain and at night.
- Finding oneself positioned in the wrong lane-especially a "turn only" lane-during an intersection approach, due to poor visibility (maintenance) of pavement markings or the obstruction of roadside signs designed to inform drivers of intersection traffic patterns.
- Difficulty at the end of an auxiliary (right) turn lane in seeing potential conflicts well and quickly enough to smoothly merge with adjacent-lane traffic.
- Merging with adjacent-lane traffic at a pavement width reduction, when the lane drop occurs near (i.e., within 150 m [500 ft]) an intersection.

Finally, the analysis by Council and Zegeer (1992) included an examination of vehicle-pedestrian crashes and the collision types in which older pedestrians were overinvolved. The results showed older pedestrians to be overrepresented in both right- and left-turn crashes. The young-elderly (ages 65–74) were most likely to be struck by a vehicle turning right, whereas the old-elderly (age 75 and older) were more likely to be struck by a left-turning vehicle.

This section will provide recommendations for 17 different design elements in order to accommodate the needs and enhance the performance of road users with age-related diminished capabilities as they approach and negotiate intersections: A. intersecting angle (skew); B. receiving lane (throat) width for turning operations; C. channelization; D. intersection sight-distance requirements; E. offset (single) left-turn lane geometry, signing, and delineation; F. edge treatments/delineation of curbs, medians, and obstacles; G. curb radius; H. traffic control for left-turn movements at signalized intersections; I. traffic control for right-turn/right-turn-on-red (RTOR) movements at signalized intersections; J. street-name signing; K. one-way/wrong-way signing; L. stop- and yield-controlled intersection signing; M. devices for lane assignment on intersection approach; N. traffic signals; O. fixed lighting installations; P. pedestrian crossing design, operations, and control; and Q. roundabouts.

Recommendations by Design Element

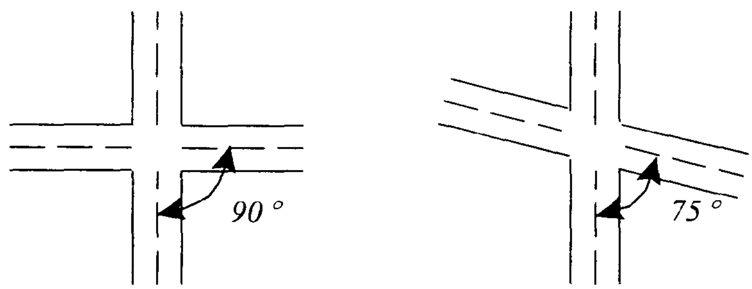
A. Design Element: Intersecting Angle (Skew)

AASHTO:1
ICG:1
ITE:1

(1) In the design of new facilities or redesign of existing facilities where right-of-way is not restricted, all intersecting roadways should meet at a 90-degree angle.

ITE:2

(2) In the design of new facilities or redesign of existing facilities where right-of-way is restricted, intersecting roadways should meet at an angle of not less than 75 degrees.



ITE:4
MUTCD:3

(3) At skewed intersections where the approach leg to the left intersects the driver's approach leg at an angle of less than 75 degrees, the prohibition of right turn on red (RTOR) is recommended [see Recommendation I(3)].

The rationale and supporting evidence for these recommendations can be found beginning on page 69 of this Handbook.

B. Design Element: Receiving Lane (Throat) Width for Turning Operations

ICG:2
ITE:2

(1) A minimum receiving lane width of 3.6 m (12 ft) is recommended, accompanied, wherever practical, by a shoulder of 1.2 m (4 ft) minimum width.

The rationale and supporting evidence for this recommendation can be found beginning on page 72 of this Handbook.

C. Design Element: Channelization

AASHTO:4
ICG:4
ITE:4
MUTCD:4

(1) **Raised channelization** with sloping curbed medians is recommended over **channelization accomplished** through the use of pavement markings **(flush) for the** following operating conditions:

(1a) **Left-** and right-turn lane treatments at intersections on all roadways with operating speeds of less than 65 km/h (40 mi/h).

(1b) **Right-turn** treatments on roadways with operating speeds equal to or greater than 65 km/h (40 mi/h).

MUTCD:4

(2) **Where** raised channelization is implemented at intersections, it is **recommended** that median and island curb sides and curb horizontal **surfaces be treated** with retroreflectorized markings and be maintained **at a minimum luminance** contrast level* as follows:

(2a) ***With*** overhead lighting, a contrast of at least 2.0 is recommended.

(2b) W*ithout* overhead lighting, a contrast of at least 3.0 is recommended.

Contrast should be calculated according to this formula:

$$luminance\ (L)\ contrast = \frac{L_{treatment} - L_{pavement}}{L_{pavement}}$$

* Luminance is the amount of light reflected from a surface to the eye of a driver. This is different from retroreflectivity, which is a property of a material. While increasing retroreflectivity generally results in higher luminance, brightness-especially at night-may vary greatly for the same target, depending on such factors as the location and intensity of its source of illumination and the angle at which a driver views it. It is the apparent brightness (more accurately, "luminance contrast") of a target in its surroundings, under representative viewing conditions, that determines its visibility (detectability) and is the critical predictor of a safe driver response. Since nighttime visibility of roadway features is most problematic for older drivers, the contrast calculation for this design element should be based on nighttime luminance measures; these should be obtained under low-beam headlight illumination from a passenger vehicle at a 5-s preview distance upstream of the intersection. Direct readings of the luminance of a surface can be obtained with a hand-held light meter that has a through-the-lens viewing system to enable accurate targeting of the design element. The luminance measurements of the target and surrounding area may be obtained from any location judged to be in the line of sight of the driver at the 5-s preview distance.

The rationale and supporting evidence for these recommendations can be found beginning on page 74 of this Handbook.

C. *Design Element: Channelization* (continued)

AASHTO:4

(3) If right-turn channelization is present at an intersection, an acceleration lane providing for the acceleration characteristics of passenger cars as delineated in AASHTO specifications (1994) is recommended.

ICG:4

(4) The use of sloping curbs rather than barrier curbs for channelization is recommended, except where the curbs surround a pedestrian refuge area or are being used for access control.

AASHTO:1
ICG:3
MUTCD:1

(5) If right-turn channelization is present and pedestrian traffic may be expected based on surrounding land use, it is recommended that an adjacent pedestrian refuge island conforming to *MUTCD* (FHWA, 2000) and AASHTO (1994) specifications be provided.

AASHTO:4
ICG:4

(6) To reduce unexpected midblock conflicts with opposing vehicles, the use of channelized left-turn lanes in combination with continuous raised-curb medians is recommended instead of center, two-way, left-turn lanes (TWLTL) for new construction or reconstruction where average daily traffic volumes exceed 20,000 vehicles per day, or for remediation where there is a demonstrated crash problem, or wherever a need is demonstrated through engineering study.

The rationale and supporting evidence for these recommendations can be found beginning on page 74 of this Handbook.

D. Design Element: Intersection Sight-Distance Requirements

AASHTO:4

(1) Where determinations of intersection sight-distance requirements for any intersection maneuver (turn left, turn right, crossing) that is performed by a driver on either a major *or* a minor road incorporate a perception-reaction time (PRT) component, it is recommended that a PRT value of no less than 2.5 s be used to accommodate the slower decision times of older drivers.

AASHTO:4

(2) Where determinations of intersection sight-distance requirements for a left-turn maneuver from a major roadway by a stopped passenger car are based on a gap model (see NCHRP Report 383), it is recommended that a gap of no less than 8.0 s, plus 0.5 s for each additional lane crossed by the turning driver, be used to accommodate the slower decision times of older drivers.

The rationale and supporting evidence for these recommendations can be found beginning on page 79 of this Handbook.

E. Design Element: Offset (Single) Left-Turn Lane Geometry, Signing, and Delineation

AASHTO:4
ICG:4
ITE:4

(1) Unrestricted sight distance (achieved through positive offset of opposing left-turn lanes) is recommended whenever possible, for new or reconstructed facilities. [See figure under Recommendation (3) below.] This will provide a margin of safety for older drivers who, as a group, do not position themselves within the intersection before initiating a left turn.

AASHTO:4
ICG:4
ITE:4

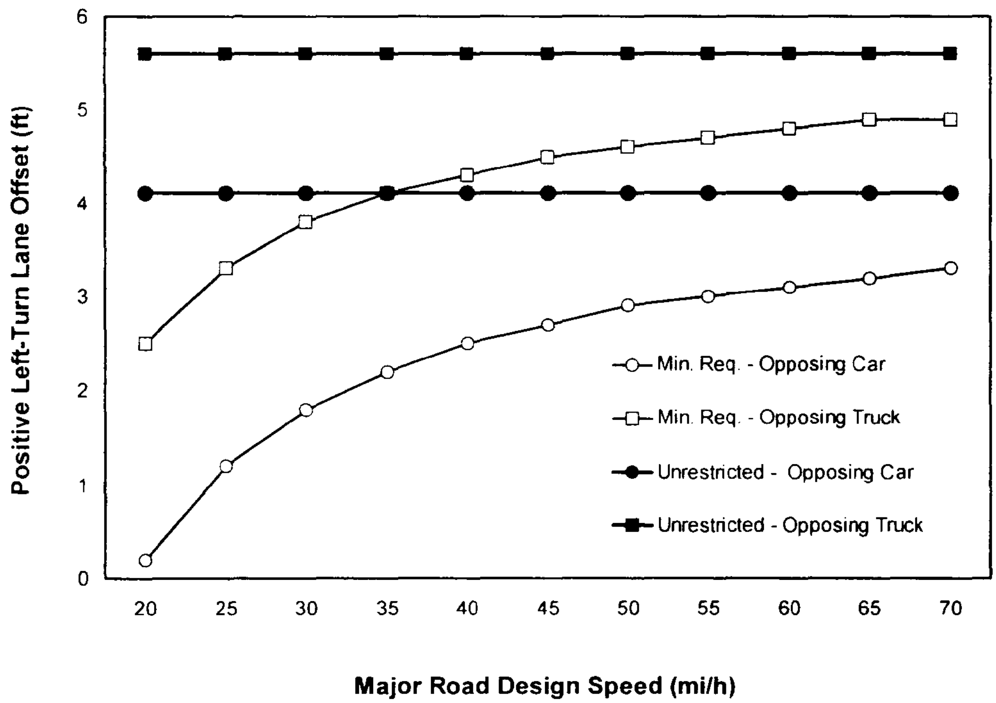
(2) At intersections where engineering judgment indicates a high probability of heavy trucks as the opposing turn vehicles during normal operations, the offsets required to provide unrestricted sight distance for opposing left-turn trucks should be used for new or reconstructed facilities. [See figure under Recommendation (3).]

The rationale and supporting evidence for these recommendations can be found beginning on page 94 of this Handbook.

E. Design Element: Offset (Single) Left-Turn Lane Geometry, Signing, and Delineation (continued)

AASHTO:4
ICG:4
ITE:4

(3) Where the provision of unrestricted sight distance is not feasible, positive left-turn lane offsets are recommended to achieve minimum required sight distances, which vary according to (major) roadway design speed and type of opposing vehicle (passenger car or heavy truck). For left-turning traffic that must yield to opposing traffic on a major roadway, the recommended offset values to achieve minimum required sight distances* are as indicated in the figure below:



1 ft=0.305 m
1 mi/h=1.61 km/h

* The functions graphed above are yielded by computations using *either* a modified AASHTO Intersection Sight Distance (ISD) formula with PRT equal to 2.5 s *or* by gap model calculations with G equal to 8.0 s plus 0.5 s for each additional lane crossed by a turning (passenger car) driver.

The rationale and supporting evidence for these recommendations can be found beginning on page 94 of this Handbook.

E. Design Element: Offset (Single) Left-Turn Lane Geometry, Signing, and Delineation (continued)

(4) At intersections where the left-turn lane treatment results in channelized offset left-turn lanes (e.g., a parallel or tapered left-turn lane between two medians), the following countermeasures are recommended to reduce the potential for wrong-way maneuvers by drivers turning left from a stop-controlled, intersecting minor roadway:

ITE:4
MUTCD:4

(4a) In the implementation of DIVIDED HIGHWAY CROSSING signs, and WRONG WAY, DO NOT ENTER, KEEP RIGHT, and ONE WAY signs at the intersection, as per *MUTCD* (FHWA, 2000) specifications, oversized signs (sizes larger than *MUTCD*-specified standard sizes for conventional roadways) are recommended.

MUTCD:1

(4b) It is recommended that the signs listed in Recommendation (4a) above be fabricated using retroreflective sheeting that provides for high retroreflectance overall, particularly at the widest available observation angles, to provide increased sign conspicuity and legibility for older drivers.

MUTCD:1

(4c) Retroreflective lane-use arrows for channelized left-turn lanes are recommended.

MUTCD:3

(4d) Retroreflective pavement marking extensions of the center line that scribe a path through the turn are recommended, except where extensions for opposing movements cross, to reduce the likelihood of wrong-way movements.

MUTCD:3

(4e) Placement of 7.1-m- (23.5-ft-) long retroreflective wrong-way arrows in the through lanes is recommended for wrong-way traffic control at locations determined to have a special need, as specified in the *MUTCD* (FHWA, 2000), sections 2A.24, 3B.19, and 2E-50.

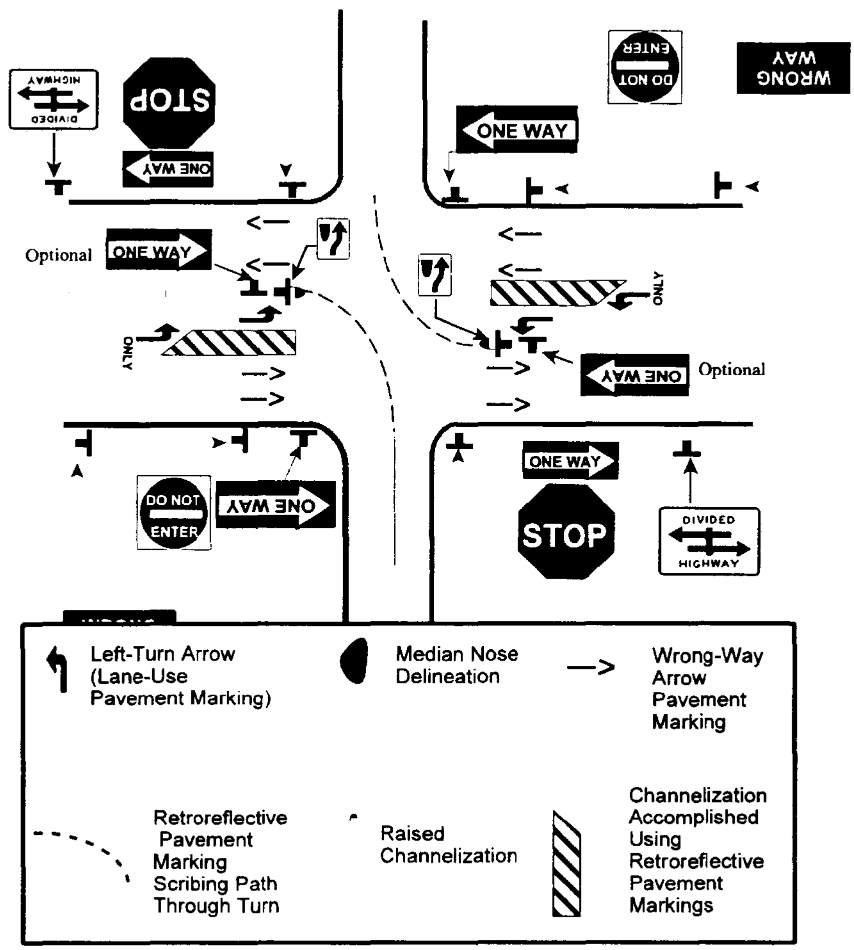
AASHTO:1
MUTCD:2

(4f) Delineation of median noses using retroreflective treatments to increase their visibility and improve driver understanding of the intersection design and function is recommended.

The rationale and supporting evidence for these recommendations can be found beginning on page 94 of this Handbook.

E. Design Element: Offset (Single) Left-Turn Lane Geometry, Signing, and Delineation (continued)

The diagram presented below illustrates the countermeasures as described above in Handbook Recommendations E(4a)–(4f).



Recommended signing and delineation treatments for intersections with medians 9 m (30 ft) wide or wider.

[Note: Median ONE WAY signs are optional where left-turn lanes result in narrowing of the median, and engineering judgment indicates a potential for motorist confusion.]

The rationale and supporting evidence for these recommendations can be found beginning on page 94 of this Handbook.

F. Design Element: Treatments/Delineation of Edgelines, Curbs, Medians, and Obstacles

MUTCD:4
RLH:4

(1) It is recommended that a minimum in-service luminance contrast* level between the marked edge of the roadway and the road surface be maintained as follows:

(1a) At intersections *with* overhead lighting, a contrast of 2.0 or higher is recommended.

(1b) At intersections *without* overhead lighting, a contrast of 3.0 or higher is recommended.

Contrast should be calculated according to this formula:

$$\textit{luminance (L) contrast} = \frac{L_{stripe} - L_{pavement}}{L_{pavement}}$$

* See advisory comments pertaining to luminance measurement in Recommendation IC (2).

AASHTO:1
MUTCD:2

(2) It is recommended that all curbs at intersections (including median islands and other raised channelization) be delineated on their vertical face and at least a portion of the top surface, in addition to the provision of a marked edgeline on the road surface.

The rationale and supporting evidence for these recommendations can be found beginning on page 101 of this Handbook.

G. *Design Element: Curb Radius*

AASHTO:1
ICG:1

(1) Where roadways intersect at 90 degrees and are joined with a simple radius curve, a corner curb radius in the range of 7.5 m to 9 m (25 ft to 30 ft) is recommended as a tradeoff to: (a) facilitate vehicle turning movements, (b) moderate the speed of turning vehicles, and (c) avoid unnecessary lengthening of pedestrian crossing distances, except where precluded by high volumes of heavy vehicles.

AASHTO:4
ICG:4
ITE:4

(2) When it is necessary to accommodate turning movements by heavy vehicles, the use of offsets, tapers, and compound curves is recommended to minimize pedestrian crossing distances.

The rationale and supporting evidence for these recommendations can be found beginning on page 105 of this Handbook.

H. *Design Element: Traffic Control for Left-Turn Movements at Signalized Intersections*

ICG:4
ITE:4
MUTCD:4

(1) The use of protected-only operations is recommended, except when, based on engineering judgment, an unacceptable reduction in capacity will result.

ITE:4
MUTCD:4

(2) To reduce confusion during an intersection approach, the use of a separate signal face to control turning phase (versus through) movements is recommended for all operating modes.

ITE:4
MUTCD:4

(3) Consistent use of the R10-12 sign, LEFT TURN YIELD ON GREEN ●, during protected-permitted operations is recommended, with overhead placement preferred at the intersection.

The rationale and supporting evidence for these recommendations can be found beginning on page 109 of this Handbook.

H. Design Element: Traffic Control for Left-Turn Movements at Signalized Intersections (continued)

AASHTO:3
MUTCD:4

(4) Where practical, the use of a redundant upstream R10-12 sign (i.e., in addition to the R10-12 sign adjacent to the signal face) is recommended to advise left-turning drivers of permitted signal operation. It is also recommended that the sign be displayed at a 3-s preview distance before the intersection, or at the beginning of the left-turn lane, as per engineering judgment, accompanied by a supplemental plaque bearing the message, AT SIGNAL. [See time-speed-distance table on page 5.]

ITE:2
MUTCD:2

(5) A leading protected left-turn phase is recommended wherever protected left-turn signal operation is implemented (as opposed to a lagging protected left-turn phase).

MUTCD:4

(6) To eliminate confusion about the meaning of the red arrow indication, it is recommended that the steady green arrow for protected-only left-turn operations terminate to a yellow arrow, then a steady circular red indication (instead of a red arrow).

AASHTO:4
ITE:1
MUTCD:1

(7) Where minimum sight-distance requirements as per recommendations for Design Element D are not practical to achieve through geometric redesign/reconstruction, or where a pattern of permitted left-turn crashes occurs, it is recommended that permitted left turns be eliminated and protected-only left-turn operations be implemented.

The rationale and supporting evidence for these recommendations can be found beginning on page 109 of this Handbook.

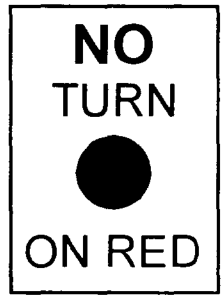
I. Design Element: Traffic Control for Right-Turn/RTOR Movements at Signalized Intersections

ITE:4
MUTCD:4

(1) It is recommended that a steady circular red indication be used at signalized intersections where a right turn on red is prohibited, instead of a red arrow indication.

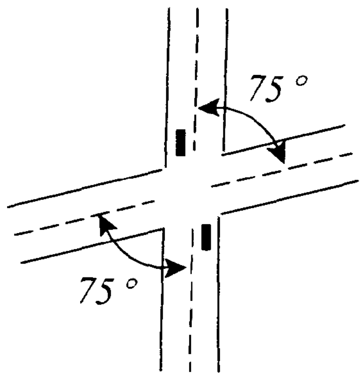
ITE:4
IEC: requires FHWA permission

(2) It is recommended that at signalized intersections where a right turn on red is prohibited, a supplemental NO TURN ON RED sign, using the design shown at right, be placed on the overhead mast arm and at a location on either the near or opposite side of the intersection where, per engineering judgment, it will be most conspicuous.



ITE:4
MUTCD:3

(3) At skewed intersections where the approach leg to the left intersects the driver's approach leg at an angle of less than 75 degrees (as indicated below), the prohibition of right turn on red (RTOR) is recommended.



MUTCD:4

(4) The posting of (black on white) signs with the legend TURNING TRAFFIC MUST YIELD TO PEDESTRIANS is recommended wherever engineering judgment indicates a clear potential for right-turning vehicles to come into conflict with pedestrians who are using the crosswalk for permitted crossing movements [shown in IP (5)].

The rationale and supporting evidence for recommendations (1)-(3) can be found beginning on page 118 of this Handbook, and on page 162 for recommendation (4).

J. Design Element: Street-Name Signing

MUTCD:4

(1) To accommodate the reduction in visual acuity associated with increasing age, a minimum letter height of 150 mm (6 in) is recommended for use on post-mounted street-name signs (*MUTCD* sign number D3) on all roads where the posted speed limit exceeds 40 km/h (25 mi/h).

MUTCD:4

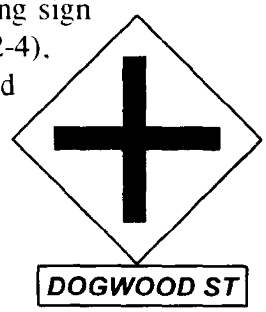
(2) The use of overhead-mounted street-name signs with mixed-case letters is recommended at major intersections as a supplement to post-mounted street-name signs. Minimum letter heights of 200-mm (8-in) uppercase letters and 150-mm (6-in) lowercase letters are recommended at major intersections with approach speeds of 56 km/h (35 mi/h) or less. At major intersections with approach speeds greater than 56 km/h (35 mi/h), the minimum letter height on street-name signs should be 250-mm (10-in) uppercase and 200-mm (8-in) lowercase letters.

MUTCD:2

(3) In the design of overhead-mounted street-name signs, the use of larger letter heights will require a larger sign panel if the Standard Alphabets for Highway Signs are used. To minimize sign-panel size while accommodating the larger letter size, it is recommended that the border be eliminated on street-name signs when using Standard Alphabets.

MUTCD:4

(4) 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 an advance street-name plaque (W16-8), as shown, using 200-mm (8-in) black letters on a yellow sign panel.



The rationale and supporting evidence for these recommendations can be found beginning on page 122 of this Handbook.

J. Design Element: Street-Name Signing (continued)

MUTCD:1

(5) 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.

MUTCD:4

(6) 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, as shown below:

Or, a two-line sign format may be used to address support and wind load issues:

MUTCD:1

(7) For post-mounted street-name signs installed at intersections in areas of intensive land use, complex design features, and heavy traffic, it is recommended that retroreflective sheeting that provides for high retroreflectance overall, particularly at the widest available observation angles, be used to provide increased sign conspicuity and legibility for older drivers.

The rationale and supporting evidence for these recommendations can be found beginning on page 122 of this Handbook.

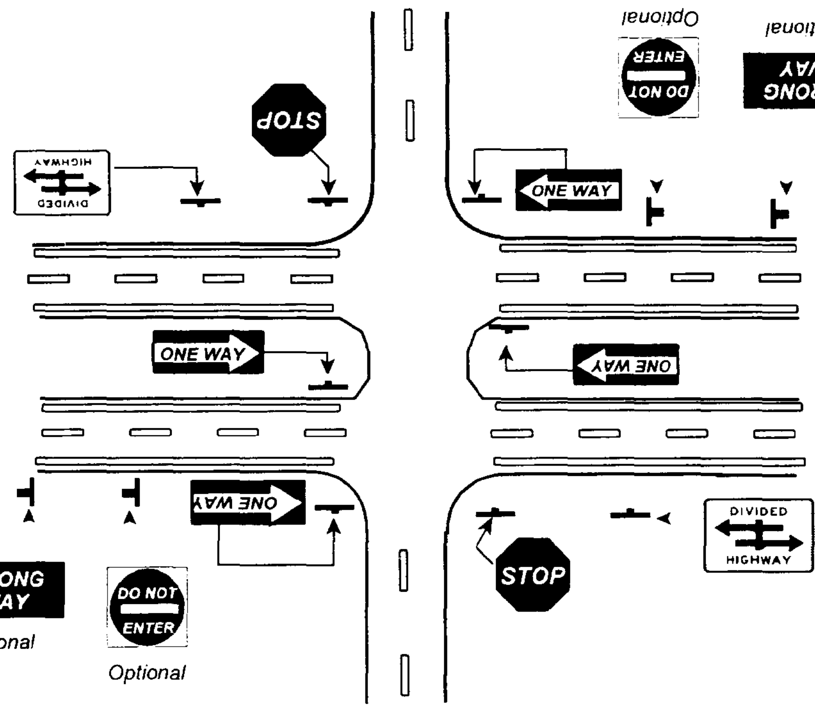
K. Design Element: One-Way/Wrong-Way Signing

MUTCD:1

(1) It is recommended that divided highways be consistently signed as shown in the configuration diagrammed below; use of the DIVIDED HIGHWAY CROSSING sign (R6-3) is the recommended practice, pending new treatments that are demonstrated through research to provide improved comprehensibility to motorists.

MUTCD:1

(2) For divided highways with median widths less than 9 m (30 ft), the use of four ONE WAY signs is recommended, located in the left median and far-right corner of the intersection, as shown in the configuration diagrammed below.



Recommended signing configuration for medians less than 9 m (30 ft).

The rationale and supporting evidence for these recommendations can be found beginning on page 131 of this Handbook.

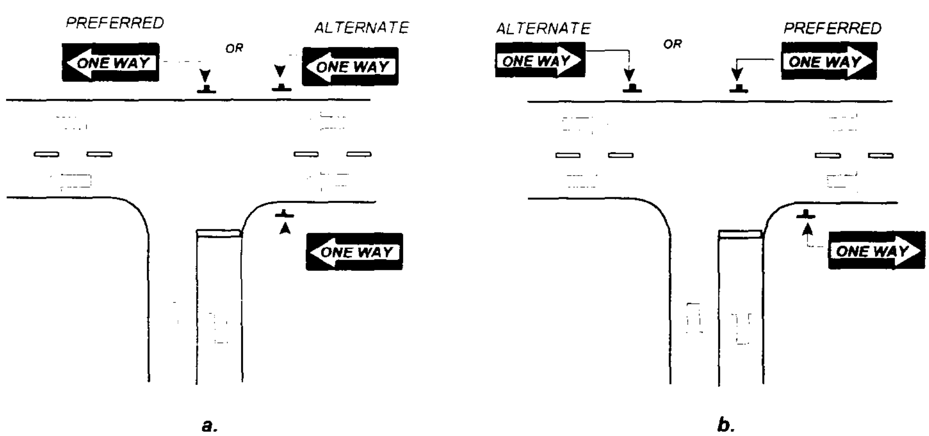
K. *Design Element: One-Way/Wrong-Way Signing* (continued)

MUTCD:3

(3) For medians ranging from 9 to 13 m (30 to 42 ft) wide, or where offset left-turn lanes are used with any median width, the use of six ONE WAY signs is recommended, as diagrammed in Recommendation (4) of Design Element E (see page 20).

MUTCD:4

(4) For T-intersections, the use of a near-right-side ONE WAY sign and a far-side ONE WAY sign is recommended; the preferred placement for the far-side sign is opposite the extended centerline of the approach leg as shown in *MUTCD* figure 2A-6 (FHWA, 2000). Where the preferred far-side location is not feasible (e.g., because of blockage, distracting far-side land use, or an excessively wide approach leg), engineering judgment should be applied to select the most conspicuous alternate location for a driver who has not yet initiated the wrong-way turning maneuver (see diagram below).

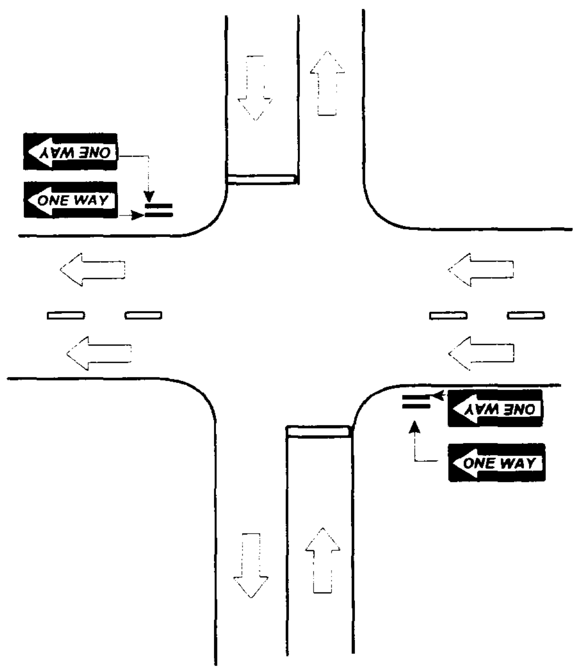


The rationale and supporting evidence for these recommendations can be found beginning on page 131 of this Handbook.

K. *Design Element: One-Way/Wrong-Way Signing* (continued)

MUTCD:4

(5) For the intersection of a one-way street with a two-way street, ONE WAY signs placed at the near-right/far-left locations are recommended, regardless of whether there is left-to-right or right-to-left traffic (see diagram below).



AASHTO:4
MUTCD:4

(6) As a general practice, the use of DO NOT ENTER and WRONG WAY signs is recommended at locations where the median width is 9 m (30 ft) and greater. Consideration should also be given to the use of these signs for median widths narrower than 9 m (30 ft), where engineering judgment indicates a special need.

The rationale and supporting evidence for these recommendations can be found beginning on page 131 of this Handbook.

L. Design Element: Stop- and Yield-Controlled Intersection Signing

Recommendations to improve the safe use of intersections by older drivers, where the need for stop control or yield control has already been determined, include the following:

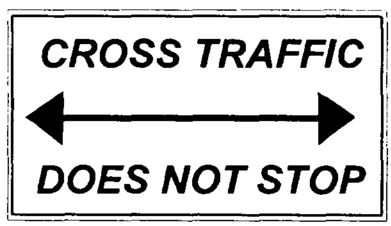
MUTCD:1 (1) The use of standard size (750-mm [30-in]) STOP (R1-1) and standard size (900-mm [36-in]) YIELD (R1-2) signs, *as a minimum*, is recommended wherever these devices are implemented, with the option of using larger R1-1 (900-mm [36-in] or 1200-mm [48-in]) signs where engineering judgment indicates that greater emphasis or visibility is required.

ITE:4
MUTCD:4 (2) A *minimum* sign background (red area) retroreflectivity level (i.e., coefficient of retroreflection [R_A]) below which a need for sign replacement is indicated, is recommended for STOP (R1-1) and YIELD (R1-2) signs as follows:

(2a) 12 cd/lux/m^2 for roads with operating speeds lower than 65 km/h (40 mi/h).

(2b) 24 cd/lux/m^2 for roads with operating speeds of 65 km/h (40 mi/h) or higher.

MUTCD:4 (3) The use of a 750-mm x 450-mm (30-in x 18-in) supplemental warning sign panel (W4-4p), as illustrated, mounted below the STOP (R1-1) sign, is recommended for two-way stop-controlled intersection sites selected on the basis of crash experience; where the sight triangle is restricted; and wherever a conversion from four-way stop to two-way stop operations is implemented.



AASHTO:4
ITE:4
MUTCD:4 (4) It is recommended that a STOP AHEAD sign (W3-1a) be used where the distance at which the STOP sign is visible is less than the AASHTO stopping sight distance (SSD) at the operating speed, plus an added preview distance of at least 2.5 s. [See time-speed-distance table on page 5.]

The rationale and supporting evidence for these recommendations can be found beginning on page 137 of this Handbook.

L. Design Element: Stop- and Yield-Controlled Intersection Signing (continued)

ITE:4

(5) The use of transverse pavement striping or rumble strips upstream of stop-controlled intersections where engineering judgment indicates a special need due to sight restrictions, high approach speeds, or a history of ran-stop-sign crashes is recommended.

The rationale and supporting evidence for these recommendations can be found beginning on page 137 of this Handbook.

M. Design Element: Devices for Lane Assignment on Intersection Approach

MUTCD:1

(1) The consistent overhead placement of lane-use control signs (e.g., R3-5, R3-6, R3-8) at intersections on a signal mast arm or span wire is recommended.

MUTCD:4

(2) The consistent posting of lane-use control signs plus application of lane-use arrow pavement markings at a preview distance of at least 5 s (at operating speed) in advance of a signalized intersection is recommended, regardless of the specific lighting, channelization, or delineation treatments implemented at the intersection. [See time-speed-distance table on page 5.] Signs should be mounted overhead wherever practical.

The rationale and supporting evidence for these recommendations can be found beginning on page 147 of this Handbook.

N. Design Element: Traffic Signals

MUTCD:4

(1) A *maintained* performance level of 200 cd for peak intensity of a 200-mm (8-in) red signal is recommended to ensure detectability and improve conspicuity of this critical control element.

MUTCD:2

(2) To accommodate age differences in perception-reaction time, it is recommended that an all-red clearance interval be consistently implemented, with length determined according to the ITE Engineers (1992) expressions given below:

(2a) Where pedestrian traffic is prohibited, or no pedestrian crossing facilities are provided, use:

$$r = \frac{W + L}{V}$$

(2b) Where pedestrian crossing facilities are provided, use:

$$r = \frac{P + L}{V}$$

where:

- r = length of red clearance interval, to the nearest 0.1 s.
- W = width of intersection (m [ft]), measured from the near-side stop line to the far edge of the conflicting traffic lane along the actual vehicle path.
- P = width of intersection (m [ft]), measured from the near-side stop line to the far side of the farthest conflicting pedestrian crosswalk along the actual vehicle path.
- L = length of vehicle (recommended as 6 m [20 ft]).
- V = speed of the vehicle through the intersection (m/s [ft/s]).

MUTCD:4

(3) The consistent use of a backplate with traffic signals on all roads with operating speeds of 65 km/h (40 mi/h) or higher is recommended. The use of a backplate with signals on roads with operating speeds lower than 65 km/h (40 mi/h) is also recommended where engineering judgment indicates a need due to the potential for sun glare problems, site history, or other variables.

The rationale and supporting evidence for these recommendations can be found beginning on page 150 of this Handbook.

O. Design Element: Fixed Lighting Installations

AASHTO:4
MUTCD:4
RLH:4

(1) Wherever feasible, fixed lighting installations are recommended as follows:

(1a) Where the potential for wrong-way movements is indicated through crash experience or engineering judgment.

(1b) Where twilight or nighttime pedestrian volumes are high.

(1c) Where shifting lane alignment, turn-only lane assignment, or a pavement-width transition forces a path-following adjustment at or near the intersection.

RLH:4

(2) Regular cleaning of lamp lenses, and lamp replacement when output has degraded by 20 percent or more of peak performance (based on hours of service and manufacturer's specifications), are recommended for all fixed lighting installations at intersections.

The rationale and supporting evidence for these recommendations can be found beginning on page 158 of this Handbook.

P. Design Element: Pedestrian Crossing Design, Operations, and Control

AASHTO:2
ICG:2
MUTCD:2

(1) To accommodate the shorter stride and slower gait of less capable (15th percentile) older pedestrians, and their exaggerated "start-up" time before leaving the curb, pedestrian control-signal timing based on an assumed walking speed of 0.85 m/s (2.8 ft/s) is recommended.

The rationale and supporting evidence for these recommendations can be found beginning on page 162 of this Handbook.

P. Design Element: Pedestrian Crossing Design, Operations, and Control
(continued)

AASHTO:4
ICG:4
ITE:4
MUTCD:4

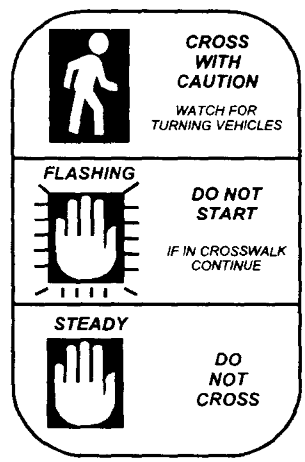
(2) For pedestrian crossings where the right-turn lane is channelized, it is recommended that:

(2a) An adjacent pedestrian refuge island conforming to *MUTCD* (FHWA, 2000) and AASHTO (1994) specifications be provided.

(2b) If a crosswalk is within the channelized area, it should be located as close as possible to the approach leg to maximize the visibility of pedestrians before drivers are focused on scanning for gaps in traffic on the intersecting roadway.

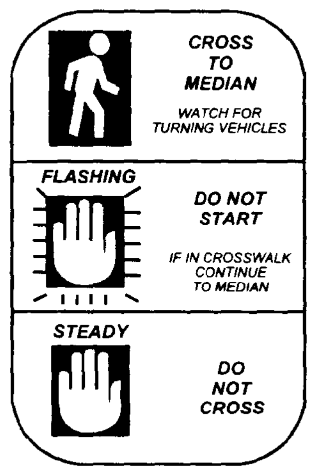
IEC: requires FHWA permission

(3) It is recommended that a placard explaining pedestrian control-signal operations and presenting a warning to watch for turning vehicles be posted at the near corner of all intersections with a pedestrian crosswalk, using the design shown.



IEC: requires FHWA permission

(4) It is recommended that, at intersections where pedestrians cross in two stages using a median refuge island, the placard depicted in Recommendation P(3) be placed on the median refuge island, and that a placard modified as shown be placed on the near corner of the crosswalk.

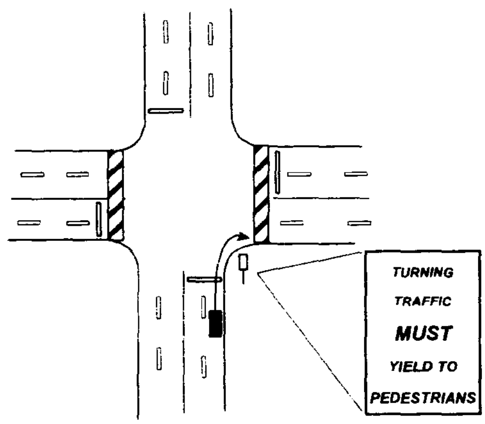


The rationale and supporting evidence for these recommendations can be found beginning on page 162 of this Handbook.

P. Design Element: Pedestrian Crossing Design, Operations, and Control (continued)

MUTCD:4

(5) The posting of (black on white) signs with the legend TURNING TRAFFIC MUST YIELD TO PEDESTRIANS is recommended wherever engineering judgment indicates a clear potential for right-turning vehicles to come into conflict with pedestrians who are using the crosswalk for permitted crossing movements (shown below).



MUTCD:4

(6) At intersections with high pedestrian volumes, high turning-vehicle volumes, and no turn on red (NTOR) control for traffic moving parallel to a marked crosswalk, a leading pedestrian interval (LPI) timed to allow slower walkers to cross at least one moving lane of traffic is recommended to reduce conflicts between pedestrians and turning vehicles. The length of the LPI, which should be at least 3 s, may be calculated using the formula:

$$LPI = (ML + PL)/2.8$$

where: LPI = seconds between onset of the WALK signal for pedestrians and the green indicator for vehicles.

ML = width of moving lane in ft.

PL = width of parking lane (if any) in ft.

2.8* = walking speed in ft/s.

* 2.8 ft/s = 0.85 m/s

The rationale and supporting evidence for these recommendations can be found beginning on page 162 of this Handbook.

Q. Design Element: Roundabouts

Recommendations for preferred practices when a State or local highway authority has determined through engineering study to install a modern roundabout during construction or reconstruction of an intersection include the following (see the figure on the following page that depicts basic geometric elements, from *Roundabouts: An Informational Guide*):

RND:1 (1) Whenever practical, it is recommended that roundabout installations be limited to one-lane entrances and exits, and one lane of circulating traffic, with the inscribed circle diameter limited to approximately 30 m (100 ft).

RND:1 (2) It is recommended that pedestrian crossings at single-lane roundabouts be set back a minimum of 7.5 m (25 ft) behind the yield line.

RND:1 (3) To control for wrong-way movements, calm traffic, and provide a pedestrian refuge for all roundabout categories, it is recommended that raised splitter islands be used, as opposed to pavement markings, to delineate the channelization. The pedestrian crosswalk area should be designed at street level (crosswalk cut through splitter island).

The rationale and supporting evidence for these recommendations can be found beginning on page 172 of this Handbook.

Q. Design Element: Roundabouts (continued)

RND:4

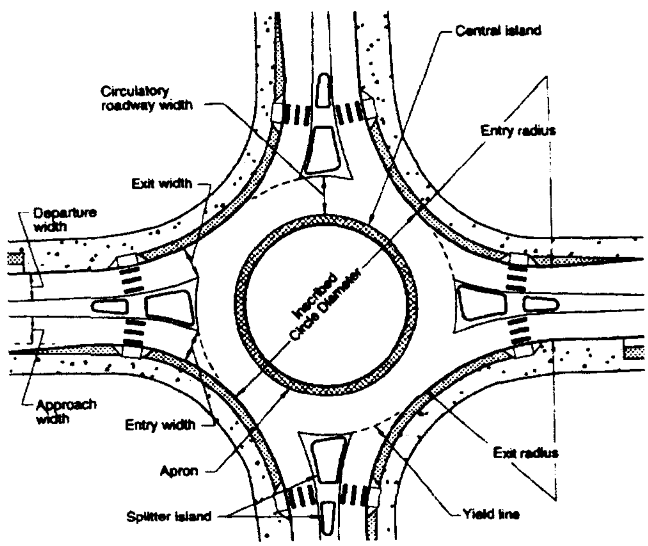
(4) To enhance the conspicuity of roundabouts in all categories, it is recommended that the sides and tops of curbs on the splitter islands and the central island be treated with retroreflective markings, and be maintained at a minimum luminance contrast level* as follows:

(4a) At roundabouts *with* overhead lighting, a contrast of 2.0 or higher is recommended.
(4b) At roundabouts *without* overhead lighting, a contrast of 3.0 or higher is recommended.

Contrast should be calculated according to this formula:

$$luminance\ (L)\ contrast = \frac{L_{stripe} - L_{pavement}}{L_{pavement}}$$

* See advisory comments pertaining to luminance measurement in Recommendation IC (2).



Basic geometric elements of a roundabout.
Source: *Roundabouts: An Informational Guide* (FHWA, 2000)

The rationale and supporting evidence for these recommendations can be found beginning on page 172 of this Handbook.

II. INTERCHANGES (GRADE SEPARATION)

Background and Scope of Handbook Recommendations

Overall, freeways are characterized by the highest safety level (lowest fatality rates) when compared with other types of highways in rural and urban areas (American Automobile Association Foundation for Traffic Safety, 1995). At the same time, freeway interchanges have design features that have been shown to result in significant safety and operational problems. Taylor and McGee (1973) reported more than 20 years ago that erratic maneuvers are a common occurrence at freeway exit ramps, and that the number of crashes there is four times greater than at any other freeway location. Two decades later, Lunenfeld (1993) reiterated that most freeway crashes and directional uncertainty occur in the vicinity of interchanges.

Distinct patterns in the occurrence of freeway interchange crashes emerge in studies that look specifically at driver age. Staplin and Lyles (1991) conducted a statewide (Michigan) analysis of the crash involvement ratios and types of violations for drivers in four age groups: age 76 and older; ages 56 to 75; ages 27 to 55; and age 26 and younger. Using induced-exposure methods to gauge crash involvement levels, this analysis showed that drivers over age 75 were overrepresented as the driver at fault in merging and weaving crashes near interchange ramps. With respect to violation types, the older driver groups were cited most frequently for failing to yield and for improper use of lanes. Similarly, Harkey, Huang, and Zegeer's study (1996) of the precrash maneuvers and contributing factors in older driver freeway crashes indicated that older drivers were much more likely than younger drivers to be merging or changing lanes, or passing/overtaking prior to a crash, and that older drivers' failure to yield was the most common contributing factor. These data raise concerns about the use of freeway interchanges by older drivers. Broader demographic and societal changes suggest that the dramatic growth in older driver freeway travel between 1977 and 1988 reported by Lerner and Ratté (1991) will continue and even accelerate in the years ahead.

Age differences in interchange crashes and violations may be understood in terms of driving task demands and age-related diminished driver capabilities. The exit gore area is a transitional area that requires a major change in tracking. A driver (especially in an unfamiliar location) must process a large amount of directional information during a short period of time and at high speeds, while maintaining or modifying his/her position within the traffic stream. When drivers must perform guidance and navigation tasks in close proximity, the chances increase that they will become overloaded and commit errors (Lunenfeld, 1993). Erratic maneuvers resulting from driver indecisiveness in such situations include encroaching on the gore area, and even backing up on the ramp or the through lane. When weaving actions are required, the information-processing task demands for both entry and exit maneuvers are further magnified.

On a population basis, the age-related diminished capabilities that contribute most to older drivers' difficulties at freeway interchanges include losses in vision and information-processing ability, and decreased physical flexibility in the neck and upper body. Specifically, older adults show declines in static and dynamic acuity, increased sensitivity to glare, poor night vision, and reduced contrast sensitivity (McFarland, Domey, Warren, and Ward, 1960; Weymouth, 1960;

Richards, 1972; Pitts, 1982; Sekuler, Kline, and Dismukes, 1982; Owsley, Sekuler, and Siemsen, 1983). These sensory losses are compounded by the following perceptual and cognitive deficits, the first two of which are recognized as being especially critical to safety: (1) reduction in the ability to rapidly localize the most relevant stimuli in a driving scene; (2) reduction in the ability to efficiently switch attention between multiple targets; (3) reduction in working-memory capacity; and (4) reduction in processing speed (Avolio, Kroeck, and Panek, 1985; Plude and Hoyer, 1985; Ponds, Brouwer, and van Wolffelaar, 1988; Brouwer, Ickenroth, Ponds, and van Wolffelaar, 1990; Brouwer, Waterink, van Wolffelaar, and Rothengatter, 1991). The most important physical losses are reduced range of motion (head and neck), which impairs visual search, and slowed response time to execute a vehicle-control movement, especially when a sequence of movements-such as braking, steering, and accelerating to weave and then exit a freeway-is required (Smith and Sethi, 1975; Goggin, Stelmach, and Amrhein, 1989; Goggin and Stelmach, 1990; Hunter-Zaworski, 1990; Staplin, Lococo, and Sim, 1990; Ostrow, Shaffron, and McPherson, 1992).

One result of these age-related diminished capabilities is demonstrated by a driver who waits when merging and entering freeways at on-ramps until he/she is alongside traffic, then relies on mirror views of overtaking vehicles on the mainline to begin searching for an acceptable gap (McKnight and Stewart, 1990). Exclusive use of mirrors to check for gaps, and slowing or stopping to look for a gap, increase the likelihood of crashes and have a negative effect on traffic flow. Malfetti and Winter (1987), in a critical incident study of merging and yielding problems, reported that older drivers on freeway acceleration lanes merged so slowly that traffic was disrupted, or they stopped completely at the end of the ramp instead of attempting to approach the speed of the traffic flow before entering the mainline. In a survey of 692 older drivers, 25 percent reported that they stop on a freeway entrance ramp before merging onto the highway, and 17 percent indicated that they have trouble finding a large enough gap in which to merge onto the mainline (Knoblauch, Nitzburg, and Seifert, 1997). Thirty-four percent of the "young-old" respondents (ages 50 to 72) and 26 percent of the "old-old" respondents (ages 73 to 97) responded that they wish entrance lanes were longer. In Lerner and Ratté's research (1991), older drivers in focus group discussions commented that they experienced difficulty maintaining vehicle headway because of slower reaction times, difficulty reading signs, fatigue, mobility limitations, a tendency to panic or become disoriented, and loss of daring or confidence. Merging onto the freeway was the most difficult maneuver discussed. Needed improvements identified by these older drivers included the elimination of weaving sections and short merge areas, which would facilitate the negotiation of on-ramps at interchanges. Improvements identified to ease the exit process included better graphics, greater use of sign panels listing several upcoming exits, and other methods to improve advance signing for freeway exits.

This section will provide recommendations for highway design elements in four areas to enhance the performance of diminished-capacity drivers at interchanges: A. exit signing and exit ramp gore delineation; B. acceleration/deceleration lane design features; C. fixed lighting installations; and D. traffic control devices for restricted or prohibited movements on freeways, expressways, and ramps.

Recommendations by Design Element

A. Design Element: Exit Signing and Exit Ramp Gore Delineation

ITE:4
MUTCD:4

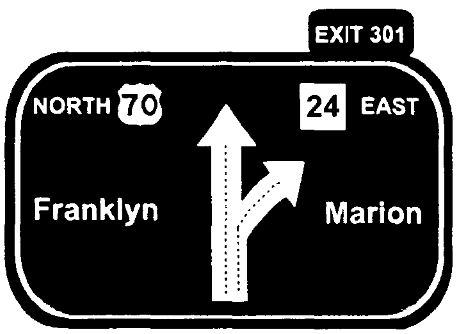
(1) The calculation of letter size requirements for signing at interchanges and on their approaches based on an assumption of *not more than* 10 m (33 ft) of legibility distance for each 25 mm (1 in) of letter height is recommended for new or reconstructed installations and at the time of sign replacement.

MUTCD:2

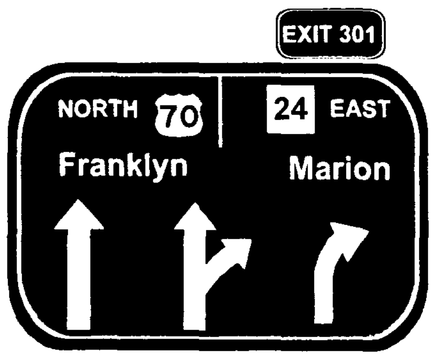
(2) To increase the reading distance of all highway destination signs, it is recommended that a mixed-case font, as presently used for overhead installations, also be used for ground-mounted signs on the side of the road (e.g., *MUTCD* sign numbers D1-1 through D1-3).

IEC: requires FHWA permission

(3) A modification of upstream diagrammatic guide signing as displayed in the *MUTCD* (figure 2E-7) is recommended for new or reconstructed installations, whereby the number of arrow shafts appearing on the sign matches the number of lanes on the roadway at the sign's location (as shown below).



Existing *MUTCD* format



Recommended alternative

The rationale and supporting evidence for these recommendations can be found beginning on page 185 of this Handbook.

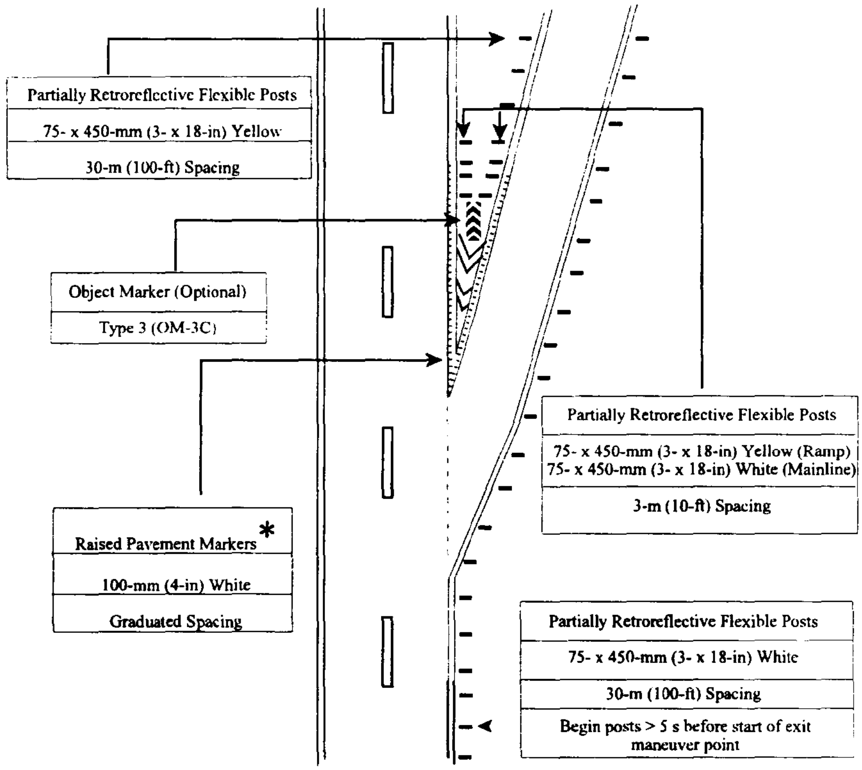
A. Design Element: Exit Signing and Exit Ramp Gore Delineation (continued)

(4) It is recommended that:

AASHTO:4
MUTCD:4

(4a) Delineation in the vicinity of the exit gore at non-illuminated and partially illuminated interchanges include, *as a minimum*, the treatments illustrated in the figure below:

Note: Figure is not to scale.



* Snowplowable raised pavement markers may be used where appropriate for conditions.

AASHTO:4
ITE:3
MUTCD:4

(4b) Where engineering judgment has identified a hazardous gore area (e.g., containing a ditch) or other special visibility need, the minimum treatment indicated in Recommendation IIA (4a) and illustrated in the figure above should be supplemented by a Type 3 object marker (OM-3C).

The rationale and supporting evidence for these recommendations can be found beginning on page 185 of this Handbook.

B. Design Element: Acceleration/Deceleration Lane Design Features

AASHTO:4

(1) 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 values for a given set of operational and geometric conditions, and assuming a 65-km/h (40-mi/h) ramp speed at the beginning of the gap search and acceptance process.

AASHTO:4

(2) A parallel versus a taper design for entrance ramp geometry is recommended.

MUTCD:3

(3) It is recommended that post-mounted delineators and/or chevrons be applied to delineate the controlling curvature on exit ramp deceleration lanes.

AASHTO:2

(4) 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).

The rationale and supporting evidence for these recommendations can be found beginning on page 197 of this Handbook.

C. Design Element: Fixed Lighting Installations

AASHTO:4
RLH:4

(1) Complete 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 (e.g., 18- to 46-m- [60- to 150-ft-] high structures with 3 to 12 luminaires per structure) per ramp is recommended, with one fixture located on the inner ramp curve near the gore, and one fixture located on the outer curve of the ramp, midway through the controlling curvature.

The rationale and supporting evidence for this recommendation can be found beginning on page 206 of this Handbook.

D. *Design Element: Traffic Control Devices for Restricted or Prohibited Movements on Freeways, Expressways, and Ramps*

MUTCD:4 (1) To increase the legibility distance of overhead lane-control signal indications for prohibited movements (red X), a double-stroke arrangement of pixels that are small (approximating a 4-mm diameter) and closely spaced (approximating 18 mm, center-to-center) is recommended.

MUTCD:4 (2) The consistent use of a 1200-mm x 750-mm (48-in x 30-in) guide sign panel with the legend FREEWAY ENTRANCE, using a minimum letter height of 200 mm (8 in) for positive guidance, as described as an option in section 2E.50 of the *MUTCD* (FHWA, 2000) and shown to the right, is recommended.

MUTCD:4 (3) 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 retroreflectorized markings and extending *as close to the crossroad as practical* without obstructing the turning path of vehicles. Where engineering judgment determines the need for the median nose to be set back from the intersection, the setback distance should be treated by a 300-mm (12-in) or wider yellow stripe bordered by yellow ceramic buttons that are touching throughout the length of the setback. In addition, it is recommended that a KEEP RIGHT (R4-7a) sign be posted on the median separator nose.

The rationale and supporting evidence for this recommendation can be found beginning on page 211 of this Handbook.

D. Design Element: Traffic Control Devices for Restricted or Prohibited Movements on Freeways, Expressways, and Ramps (continued)

(4) To meet overriding concerns for enhanced conspicuity of signing for prohibited movements, the following countermeasures are recommended where DO NOT ENTER (R5-1) and WRONG WAY (R5-1a) signs are used:

MUTCD:4

(4a) A minimum size for R5-1 of 900 mm x 900 mm (36 in x 36 in) and 1200 mm x 800 mm (48 in x 32 in) for R5-1a is recommended, with corresponding increases in letter size.

MUTCD:1

(4b) To provide increased sign conspicuity and legibility for older drivers, retroreflective fluorescent red sheeting materials that provide for high retroreflectance overall, and particularly at the widest available observation angles, are recommended.

MUTCD:4

(4c) Where engineering judgment indicates an exaggerated risk of wrong-way movement crashes, it is recommended that both the R5-1 and R5-1a signs be installed on both sides of the ramp, placed in accordance with the *MUTCD*.

IEC: requires FHWA permission

(4d) Where all other engineering options have been tried or considered, lowering sign height* to maximize brightness under low-beam headlight illumination is recommended by mounting the signs 900 mm (36 in) above the pavement (measured from the road surface to the bottom of the sign), or the lowest value above 900 mm that is practical when the presence of snow, vegetation, or other obstructions is taken into consideration.

* This does not meet the standards set by *MUTCD* Section 2A.18 or *Roadside Design Guide* Section 4.3.3, so the reasons for choosing to implement this recommendation should be clearly documented by the authorized agency.

MUTCD:3

(5) The application of 7.1-m- (23.5-ft-) long wrong-way arrow pavement markings (see *MUTCD* section 3B.19, figure 3B-22) near the terminus on all exit ramps is recommended.

The rationale and supporting evidence for this recommendation can be found beginning on page 211 of this Handbook.

III. ROADWAY CURVATURE AND PASSING ZONES

Background and Scope of Handbook Recommendations

Crashes on horizontal curves have been recognized as a considerable safety problem for many years. Crash studies indicate that roadway curves experience a higher crash rate than tangents, with rates ranging from one-and-a-half to three or four times higher than tangents (Glennon, Neuman, and Leisch, 1985; Zegeer, Stewart, Reinfurt, Council, Neuman, Hamilton, Miller, and Hunter, 1990; Neuman, 1992). Lerner and Sedney (1988) reported anecdotal evidence that horizontal curves present problems for older drivers. Also, analyses of crash data in Michigan found that older drivers were involved in crash situations on horizontal curves as a result of driving too fast for the curve or, more significantly, because they were surprised by the curve alignment (Lyles, Kane, Vanosdall, and McKelvey, 1997). In reviewing literature on driver behavior on rural road curves, Johnston (1982) reported that horizontal curves that are less than 600 m (1968 ft) in radius on two-lane rural roads, and those requiring a substantial reduction in speed from that prevailing on the preceding tangent section, were disproportionately represented among crash sites.

Successful curve negotiation depends on the choice of appropriate approach speed and adequate lateral positioning through the curve. Many studies have shown that loss-of-control crashes result from an inability to maintain a lateral position through the curve because of excessive speed, with inadequate deceleration in the approach zone. These problems, in turn, stem from a combination of factors, including poor anticipation of vehicle control requirements, induced by the driver's prior speed, and inadequate perception of the demands of the curve.

Many studies report a relationship between horizontal curvature (and the degree of curvature) and the total percentage of crashes by geometric design feature on the highways. The reasons for these crashes are related to the following inadequate driving behaviors:

- Deficient skills in negotiating curves, especially those of more than 3 degrees (Eckhardt and Flanagan, 1956).
- Exceeding the design speed on the curve (Messer, Mounce, and Brackett, 1981).
- Exceeding the design of the vehicle path (Glennon and Weaver, 1971; Good, 1978).
- Failure to maintain appropriate lateral position in the curve (McDonald and Ellis, 1975).
- Incorrect anticipatory behavior of curve speed and alignment when approaching the curve (Messer et al., 1981; Johnston, 1982).
- Inadequate appreciation of the degree of hazard associated with a given curve (Johnston, 1982).

With respect to vertical curves, design policy is based on the need to provide drivers with adequate stopping sight distance (SSD). That is, enough sight distance must exist to permit drivers to see an obstacle soon enough to stop for it under some set of reasonable worst-case conditions. The parameters that determine sight distance on crest vertical curves include the change of grade, the length of the curve, the height above the ground of the driver's eye, and the height of the obstacle to be seen. SSD is determined by the driver's reaction time, speed of the vehicle, and tire-pavement coefficient of friction. However, there is some concern with the validity of the SSD model that has been in use for more than 50 years. Current practice assumes an obstacle height of 150 mm (6 in) and a locked-wheel, wet-pavement stop (AASHTO, 1994). Minimum lengths of crest vertical curves are based on sight distance and driver comfort. These criteria *do not* currently include adjustments for age-related effects in driving performance measures, which would suggest an even more conservative approach. At the same time, the general lack of empirical data demonstrating benefits for limited sight-distance countermeasures has led some to propose liberalization of model criteria, such as obstacle height (Neuman, 1989; Fambro, Fitzpatrick, and Koppa, 1997).

Standards and criteria for sight distance, horizontal and vertical alignment, and associated traffic control devices are based on the following driver performance characteristics: detection and recognition time, perception-reaction time, decision and response time, time to perform brake and accelerator movements, maneuver time, and (if applicable) time to shift gears. However, these values have typically been based on driving performance (or surrogate driving measures) of the entire driving population, or have been formulated from research biased toward younger (college age) as opposed to older driver groups. The models underlying these design standards and criteria therefore have not, as a rule, included variations to account for slower reaction time or other performance deficits consistently demonstrated in research on older driver response capabilities. In particular, diminished visual performance (reduced acuity and contrast sensitivity), physical capability (reduced strength to perform control movements and sensitivity to lateral force), cognitive performance (attentional deficits and declines in choice reaction time in response to unpredictable stimuli), and perceptual abilities (reduced accuracy of processing speed-distance information as required for gap judgments) combine to make the task of negotiating the highway design elements addressed in this section more difficult and less forgiving for older drivers.

This section will provide recommendations to enhance the performance of diminished-capacity drivers as they negotiate roadway curvature and passing zones, focusing on four design elements: A. pavement markings and delineation on horizontal curves; B. pavement width on horizontal curves; C. crest vertical curve length and advance signing for sight-restricted locations; and D. passing zone length, passing sight distance, and passing/overtaking lanes on two-lane highways.

Recommendations by Design Element

A. Design Element: Pavement Markings and Delineation on Horizontal Curves

MUTCD:4

(1) Recommendations for the maintained brightness of white edgelines on horizontal curves are presented in terms of measured* effective luminance contrast level (C), where:

$$Luminance\ contrast\ (C) = \frac{Luminance_{stripe} - Luminance_{pavement}}{Luminance_{pavement}}$$

* See advisory comments pertaining to luminance measurement in Recommendation IC (2).

Specifically,

(1a) On highways without median separation of opposing directions of traffic, the recommended minimum in-service contrast level for edgelines on horizontal curves is 5.0.

(1b) On highways where median barriers effectively block the drivers' view of oncoming headlights or where median width exceeds 15 m (50 ft), the recommended minimum in-service contrast level for edgelines on horizontal curves is 3.75.

MUTCD:4

(2) For horizontal curves with radii less than 1000 m (3280 ft), it is recommended that standard centerline markings be supplemented with raised pavement markers (RPMs) installed at standard spacing (i.e., 12 m [40 ft] apart), and that they be applied for a distance of 5 s of driving time (at an 85^{th} percentile speed) on the approach to the curve and continued throughout the length of the curve. [See time-speed-distance table on page 3.]

The rationale and supporting evidence for these recommendations can be found beginning on page 219 of this Handbook.

A. Design Element: Pavement Markings and Delineation on Horizontal Curves (continued)

MUTCD:4

(3) In addition to the installation of chevron alignment signs (W1-8) as specified in section 2C.10 of the *MUTCD* (FHWA, 2000), it is recommended that:

(3a) Roadside post-mounted delineation (PMDs) devices be installed at a maximum spacing (S) of 12 m (40 ft) on all horizontal curves with a radius (R) of 185 m (600 ft) or less.

(3b) The standard formula specified in *MUTCD* section 3D.4, Table 3D-1 (FHWA, 2000) be used to define roadside delineator spacing intervals for curves with radii of more than 185 m (600 ft), where:

English:

$$S = 3\sqrt{R - 50}$$

Metric:

$$S = 1.7\sqrt{R - 15}$$

Where: R=radius of curve (in feet); S=spacing on curve (in feet) — R=radius of curve (in meters); S=spacing on curve (in meters)

The rationale and supporting evidence for these recommendations can be found beginning on page 219 of this Handbook.

B. Design Element: Pavement Width on Horizontal Curves

AASHTO:4
ITE:4

(1) For horizontal curves on two-lane non-residential facilities that have ≥ 3 degrees of curvature, it is recommended that the width of the lane plus the paved shoulder be at least 5.5 m (18 ft) throughout the length of the curve (assuming AASHTO [1994] design values for superelevation and coefficient of side friction).

The rationale and supporting evidence for this recommendation can be found beginning on page 230 of this Handbook.

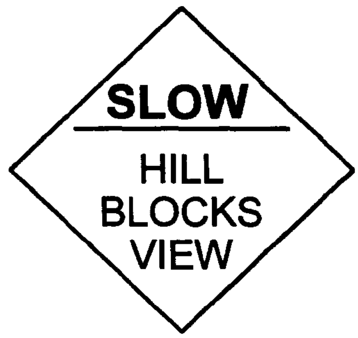
C. Design Element: Crest Vertical Curve Length and Advance Signing for Sight-Restricted Locations

MUTCD:1

(1) To accommodate the exaggerated decline among older drivers in response to unexpected hazards, it is recommended that the present criterion of 150 mm (6 in) for obstacle height on crest vertical curves be preserved in the design of new and reconstructed facilities.

IEC: requires FHWA permission

(2) Where a need has been determined for installation or replacement of a device to warn motorists that sight distance is restricted by a crest vertical curve, the message SLOW / HILL BLOCKS VIEW is recommended, using the special sign size of 900 mm x 900 mm (36 in x 36 in) as a minimum.



MUTCD:4

(3) If a signalized intersection is obscured by vertical or horizontal curvature in a manner that the signal phase becomes visible at a preview distance of 8 s or less (at operating speed), then it is recommended that the standard (W3-3) advance signal warning sign be augmented with a yellow placard bearing the black legend PREPARE TO STOP and a flashing yellow beacon interconnected with the traffic signal controller. The yellow flasher should be activated at a sufficient interval prior to the onset of the yellow signal phase and sustained after the onset of the green signal phase to take into account the end of queues experienced during peak traffic conditions, as determined through engineering study. [See time-speed-distance table on page 3.]

The rationale and supporting evidence for these recommendations can be found beginning on page 233 of this Handbook.

D. *Design Element: Passing Zone Length, Passing Sight Distance, and Passing/Overtaking Lanes on Two-Lane Highways*

AASHTO:2
ITE:2
MUTCD:2

(1) To accommodate age-related difficulties in judging gaps and longer decision-making and reaction times exhibited by older drivers, the most conservative minimum required passing sight distance (PSD) values, as determined by AASHTO (1994, table III-5), are recommended.

MUTCD:1
MUTCD:3

(2) Use of the *MUTCD* (FHWA, 2000) special-size (1200-mm x 1600-mm x 1600-mm [48-in x 64-in x 64-in]) NO PASSING ZONE pennant (W14-3), or the standard size (900 mm x 1200 mm x 1200 mm [36 in x 48 in x 48 in]) using fluorescent yellow retroreflective sheeting, as a high-conspicuity supplement to conventional centerline pavement markings at the beginning of no passing zones is recommended.

AASHTO:4
ITE:4

(3) To the extent feasible for new or reconstructed facilities, the implementation of passing/overtaking lanes (in each direction) at intervals of no more than 5 km (3.1 mi) is recommended.

The rationale and supporting evidence for these recommendations can be found beginning on page 238 of this Handbook.

IV. CONSTRUCTION/WORK ZONES

Background and Scope of Handbook Recommendations

Highway construction and maintenance zones deserve special consideration with respect to older driver needs because of their strong potential to violate driver expectancy. Alexander and Lunenfeld (1986) properly emphasized that driver expectancy is a key factor affecting the safety and efficiency of all aspects of the driving task. Consequently, it is understandable that crash analyses consistently show that more crashes occur on highway segments containing construction zones than on the same highway segments before the zones were implemented (Juergens, 1972; Graham, Paulsen, and Glennon, 1977; Lisle, 1978; Nemeth and Migletz, 1978; Paulsen, Harwood, and Glennon, 1978; Garber and Woo, 1990; Hawkins, Kacir, and Ogden, 1992).

Work-zone traffic control must provide adequate notice to motorists that describes the condition ahead, the location, and the required driver response. Once drivers reach a work zone, pavement markings, signing, and channelization must be conspicuous and unambiguous in providing guidance through the area. The National Transportation Safety Board (NTSB, 1992) stated that the *MUTCD* guidelines concerning signing and other work-zone safety features provide more than adequate warning for a *vigilant* driver, but may be inadequate for an inattentive or otherwise impaired driver. It is within this context that functional deficits associated with normal aging, as described below, may place older drivers at greater risk when negotiating work zones.

In a crash analysis at 20 case-study work-zone locations, among the most frequently listed contributing factors were driver attention errors and failure to yield the right-of-way (Pigman and Agent, 1990). Older drivers are most likely to demonstrate these deficits. Research on selective attention has documented that older adults respond much more slowly to stimuli that are unexpected (Hoyer and Familant, 1987), suggesting that older adults could be particularly disadvantaged by changes in roadway geometry and operations such as those found in construction zones. There is also research indicating that older adults are more likely to respond to new traffic patterns in an "automatized" fashion, resulting in more frequent driver error (Fisk, McGee, and Giambra, 1988). To respond in situations that require decisions among multiple and/or unfamiliar alternatives, with unexpected path-following cues, drivers' actions are described by *complex reaction times* that are longer than reaction times in simple situations with expected cues. In Mihal and Barrett's analysis (1976) relating simple, choice, and complex reaction time to crash involvement, only an increase in complex reaction time was associated with crashes. The relationship with driver age was most striking: the correlation between complex reaction time and crash involvement increased from $r = 0.27$ for the total analysis sample (all ages) to $r = 0.52$ when only older adults were included. Such data suggest that in situations where there is increased complexity in the information to be processed by drivers-such as in work zones-the most relevant information must be communicated in a dramatic manner to ensure that it receives a high priority by older individuals.

Compounding their exaggerated difficulties in allocating attention to the most relevant aspects of novel driving situations, diminished visual capabilities among older drivers are well documented (McFarland, Domey, Warren, and Ward, 1960; Weymouth, 1960; Richards, 1972; Pitts, 1982; Sekuler, Kline, and Dismukes, 1982; Owsley, Sekuler, and Siemsen, 1983; Wood and Troutbeck, 1994). Deficits in static and dynamic acuity and contrast sensitivity, particularly under low-luminance conditions, make it more difficult for them to detect and read traffic signs, to read variable message signs, and to detect pavement markings and downstream channelization devices. Olson (1988) determined that for a traffic sign to be noticed at night in a visually complex environment, its reflectivity must be increased by a factor of 10 to achieve the same level of conspicuity as a sign in a low-complexity environment. Mace (1988) asserted that the minimum required visibility distance-the distance from a traffic sign required by drivers in order to detect the sign, understand the situation, make a decision, and complete a vehicle maneuver before reaching a sign-is increased significantly for older drivers due to their poorer visual acuity and contrast sensitivity, coupled with inadequate sign luminance and legend size. Other age-related deficits cited by Mace (1988) include lowered driver alertness, slower detection time in complex roadway scenes due to distraction from irrelevant stimuli, increased time to understand unclear messages such as symbols, and slower decision making.

In a mail survey of 1,329 American Association of Retired Persons (AARP) members ages 50 to 97, conducted to identify older driver freeway needs and capabilities, 21 percent of the respondents indicated that they have problems with accurately judging distances in construction zones (Knoblauch, Nitzburg, and Seifert, 1997). These drivers reported additional problems in negotiating work zones, including congestion/traffic; lack of adequate warning; narrow lanes; lane closures and lane shifts; and difficulty staying in their lane.

This section will provide recommendations to enhance the performance of diminished-capacity drivers as they approach and travel through construction/work zones, keyed to five specific design elements: A. lane closure/lane transition practices; B. portable changeable (variable) message signing practices; C. channelization practices (path guidance); D. delineation of crossovers/alternate travel paths; and E. temporary pavement markings.

Recommendations by Design Element

A. Design Element: Lane Closure/Lane Transition Practices

MUTCD:4

(1) At construction/maintenance work zones on high-speed roadways (where the posted speed limit is 72 km/h [45 mi/h] or greater) and divided highways, the consistent use of a flashing arrow panel located at the taper for each lane closure is recommended.

ITE:4
MUTCD:4

(2) In implementing advance signing for lane closures as per *MUTCD* Part 6, it is recommended that:

A supplemental (portable) changeable message sign (CMS) displaying the one-page (phase) message LEFT (RIGHT, CENTER) LANE CLOSED should be placed 800 to 1600 m (2625 to 5250 ft) upstream of the lane closure taper.

or

Redundant static signs should be used, with a minimum letter height of 200 mm (8 in) and fluorescent orange retroreflective sheeting that provides high retroreflectance at the widest available observation angle, where both the first upstream sign (e.g., W20-1) and the second sign (e.g., W20-5) encountered by the driver are equipped with flashing warning lights throughout the entire time period of the lane closure.

The rationale and supporting evidence for these recommendations can be found beginning on page 243 of this Handbook.

B. Design Element: Portable Changeable (Variable) Message Signing Practices

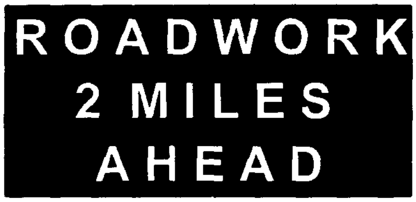
ITE:2 MUTCD:4	(1)	It is recommended that no more than two phases be used on a changeable message sign (CMS); if a message cannot be conveyed in two phases, multiple CMSs and/or a supplemental highway advisory radio message should be used.
MUTCD:4	(2)	It is recommended that each phase of a CMS message be displayed for a minimum of 3 s.
MUTCD:4	(3)	It is recommended that no more than one unit of information be displayed on a single line on a CMS, and no more than three units should be displayed for any single phase. A unit of information is one or more words that answers a specific question (e.g., *What happened? Where? What is the effect on traffic? What should the driver do?*).
ITE:4 MUTCD:4	(4)	For CMS messages split into two phases, a total of no more than four *unique* units of information should be presented.

The rationale and supporting evidence for these recommendations can be found beginning on page 253 of this Handbook.

B. Design Element: Changeable (Variable) Message Signing Practices
(continued)

ITE:4
MUTCD:4

(5) When a portable CMS is used to display a message in two phases, the problem and location statements should be displayed during phase 1 and the effect or action statement during phase 2. For example:



Phase 1

Phase 2

If legibility distance restrictions rule out a two-phase display, the use of abbreviations [as specified in the *MUTCD* (FHWA, 2000)] plus elimination of the problem statement is the recommended strategy to allow for the presentation of the entire message in one phase:

The rationale and supporting evidence for these recommendations can be found beginning on page 253 of this Handbook.

B. Design Element: Changeable (Variable) Message Signing Practices (continued)

(6) For superior legibility, it is recommended that:

MUTCD:4 (6a) Only single-stroke fonts should be used for displays of alphanumeric characters on portable CMSs with the conventional 5- x 7-pixel matrix; double-stroke fonts should be avoided.

MUTCD:4 (6b) As new portable CMSs are procured by a highway agency, the performance specifications of such devices should include a minimum character width-to-height ratio of 0.7 (complete character) and a maximum stroke width-to-height ratio of 0.13.

The rationale and supporting evidence for these recommendations can be found beginning on page 253 of this Handbook.

C. Design Element: Channelization Practices (Path Guidance)

(1) The following minimum dimensions or properties for channelizing devices used in highway work zones are recommended to accommodate the needs of older drivers:

MUTCD:4 (1a) Traffic cones-900 mm (36 in) high, with two bands of retroreflective material totaling at least 300 mm (12 in) wide for nighttime operations.

MUTCD:4 (1b) Tubular markers-1050 mm (42 in) high, with a single band of retroreflective material at least 300 mm (12-in) wide for nighttime operations.

MUTCD:1 (1c) Vertical (striped) panels-300 mm (12 in) wide.

MUTCD:4 (1d) Chevron panels (W1-8) modified in color to be used in a work zone (white on orange)-450 mm (18 in) wide and 600 mm (24 in) high.

MUTCD:2 (1e) Barricades-300-mm x 900-mm (12-in x 36-in) minimum dimensions.

MUTCD:4 (1f) Drums-450 mm x 900 mm (18 in x 36 in), with high-brightness sheeting for the orange and white retroreflective stripes (as per *MUTCD* guidelines).

The rationale and supporting evidence for these recommendations can be found beginning on page 266 of this Handbook.

C. Design Element: Channelization Practices (Path Guidance)
(continued)

MUTCD:4

(2) It is recommended that channelizing devices through work zones (in non-crossover applications) be spaced at no more than a distance in feet equal to the speed limit through the work zone in miles per hour (e.g., in 40-mi/h work zone, channelizing devices should be spaced no farther apart than 40 ft). Where engineering judgment indicates a special need for speed reduction where there is horizontal curvature or through the taper for a lane closure, spacing of channelizing devices at a distance in feet equal to no more than half of the speed limit in miles per hour is recommended (e.g., in a 40-mi/h zone, space the devices no farther apart than 20 ft).

MUTCD:4

(3) The use of side reflectors with cube-corner lenses or reflectors (facing the driver) mounted on top of concrete safety-shaped barriers and related temporary channelizing barriers is recommended, spaced (in feet) at not more than the construction zone speed limit (in miles per hour) through a work zone.

The rationale and supporting evidence for this recommendation can be found beginning on page 266 of this Handbook.

D. Design Element: Delineation of Crossovers/Alternate Travel Paths

MUTCD:1

(1) The use of positive barriers in transition zones and positive separation (channelization) between opposing two-lane traffic throughout a crossover is recommended, for intermediate- and long-term-duration work zones, for all roadway classes except residential.

MUTCD:4

(2) A minimum spacing (in feet) of one-half the construction zone speed limit (<u>in miles per hour</u>) for channelizing devices (other than concrete barriers) is recommended in transition areas and through the length of the crossover, and in the termination area downstream (where operations as existed prior to the crossover resume).

MUTCD:4

(3) The use of side reflectors with cube-corner lenses spaced (in feet) at not more than the construction zone speed limit (<u>in miles per hour</u>) on concrete channelizing barriers in crossovers (or alternately the use of retroreflective sheeting on plastic glare-control louvers [paddles] placed in crossovers) is recommended.

MUTCD:4

(4) It is recommended for construction/work zones on high-volume roadways that plastic glare-control louvers (paddles) be mounted on top of concrete channelizing barriers, when used in transition and crossover areas, at a spacing of not more than 600 mm (24 in).

The rationale and supporting evidence for these recommendations can be found beginning on page 272 of this Handbook.

E. Design Element: Temporary Pavement Markings

MUTCD:2

(1) Where temporary pavement markings shorter than the 3-m (10-ft) standard length are implemented, it is recommended that a raised pavement marker be placed at the center of the gap between successive markings.

The rationale and supporting evidence for this recommendation can be found beginning on page 277 of this Handbook.

V. HIGHWAY-RAIL GRADE CROSSINGS (PASSIVE)

Background and Scope of Handbook Recommendations

According to the Federal Railroad Administration (FRA, 1999), in 1998, there were 3,508 highway-rail grade crossing crashes, resulting in 431 fatalities and 1,303 injuries. The majority of these incidents (64 percent) occurred during the day, 31 percent occurred at night, and 5 percent occurred during dusk/dawn. Fifty-five percent of the crashes in 1998 occurred at crossings with passive controls. In a National Transportation Safety Board study (NTSB, 1998), driver error was cited as the probable cause of the crash in 49 of 60 vehicle crashes analyzed at highway grade crossings with passive controls.

Klein, Morgan, and Weiner (1994) analyzed Fatal Analysis Reporting System (FARS) data from 1975 to 1992 to determine the characteristics of drivers involved in highway-rail grade crossings and the circumstances under which such crashes occurred. This analysis indicated that drivers ages 25 to 34 are involved in the highest percentage (almost 25 percent) of all fatal rail crossing crashes, followed by drivers ages 16 to 20 (approximately 18 percent). Drivers in these age groups also show the highest involvement in all fatal crashes and all fatal intersection crashes, based on crash frequency data uncorrected for exposure. In contrast, drivers ages 65 to 74 were involved in 6.5 percent of fatal railroad crossing crashes and drivers ages 75 and older account for almost 5 percent of the railroad crossing fatalities. Again, these data do not reflect the level of exposure. However, the data show that the percentage of drivers ages 65 to 74 who are involved in fatal rail crossing crashes is slightly more than the percentage of drivers in this age group who are involved in all fatal crashes (4.6 percent) and about the same as those involved in fatal intersection crashes (6.2 percent), which is the maneuver category for which seniors are most at risk. Notably, the proportion of older drivers involved in highway-rail grade crossing crashes at night is higher than the proportion of older drivers in vehicle-involved crashes at night, suggesting special problems associated with the use of these facilities under reduced visibility conditions.

There are several age-related diminished capabilities that may make the task of safely negotiating highway-rail grade crossings more difficult for older drivers. Well-documented losses in visual acuity and contrast sensitivity with advancing age (Burg, 1967; Ball and Owsley, 1991; Ball, Owsley, Sloane, Roenker, and Bruni, 1993; Decina and Staplin, 1993) may substantially delay the detection of critical elements such as the standard crossbuck or warning symbol during a motorist's approach to a crossing, and may preclude detection of a train actually present at the crossing until impact is imminent, especially at night. While the analyses of Klein et al. (1994) paint a compelling picture of young males engaging in intentionally risky behavior as a significant component of the crash problem at rail crossings, the technical literature suggests that willful noncompliance with traffic control devices by seniors at these sites will not be a major problem-*if* they (visually) detect and comprehend the advisory, warning, and regulatory information conveyed by these devices in time to respond safely.

Expectancy also plays a role in where and when drivers look for trains, and consequently, train detection (Raslear, 1995). A driver who is familiar with a crossing and rarely or never encounters a train during the time period he or she uses the crossing is more likely to miss seeing a train than either the driver who is unfamiliar with the crossing and therefore has no expectations about train frequency, or the driver who is familiar with the crossing and frequently encounters trains during the time period that he or she crosses the tracks. Drivers who don't expect trains do not look for them. As a consequence, per train, crash rates are higher for crossings with the lowest frequency of trains (Raslear, 1995). Enhancing the conspicuity and comprehension of design elements at passive crossings, plus the use of signing that orients drivers' attention toward trains and advises drivers on the appropriate action to be taken, are thus top priorities.

Comprehension of highway-rail grade crossing traffic control devices and the performance of related information-processing tasks may be expected to pose disproportionate difficulty for older drivers. Although the crossbuck sign is a regulatory sign that serves as an implied YIELD sign, researchers consistently report that drivers do not understand the message it is intended to convey (Bridwell, Alicandri, Fischer, and Kloeppel, 1993; Fambro, Shull, Noyce, and Rahman, 1997).

Furthermore, assuming that a driver has been properly alerted to the need to search for an approaching train by design elements upstream and at the crossing, has slowed, and has begun to actively scan the tracks in each direction, the perception-reaction time (PRT) for a decision either to stop or to proceed, plus the subsequent execution of a brake or accelerator response, draw upon abilities found to slow significantly among the elderly (Staplin and Fisk, 1991; Goggin, Stelmach, and Amrhein, 1989; Stelmach, Goggin, and Amrhein, 1988). Whereas AASHTO (1994) uses a PRT of 2.5 s for calculating the sight triangle at passive grade crossings, more than a decade ago, Gordon, McGee, and Hooper (1984) recommended that a full second be added to this design value to accommodate the 85th percentile driver. With the ever-increasing number and percentage of senior drivers, the need to refocus attention on this issue is urgent.

Additional insight is provided by Leibowitz (1985), who showed that inaccurate judgment of train speed and distance may make drivers' decisions to cross hazardous, due to perceptual illusions. Most drivers are not aware of the effects of the illusions of perspective, train size, and velocity (e.g., the bigger the object, the slower it appears to be moving), and this results in unsafe crossing decisions. Kinnan (1993) states that, in most cases, the driver believes the decision to cross is a rational one; most motorists seriously underestimate the risk because they can't properly gauge the speed of the train or its distance from the crossing. This problem will only be exacerbated by the age-related decline in the ability to integrate speed and distance information, as reported by Staplin, Lococo, and Sim (1993) for the judgment of gaps at intersections.

Finally, age-related hearing loss may contribute to a failure to detect a train approaching a crossing. According to government statistics (DHHS, 1994), approximately 30 to 35 percent of people ages 65 to 75 have a hearing loss, increasing to 40 percent for persons over the age of 75. Janke (1994) reported that totally deaf males have more crashes than their non-deaf counterparts, and drivers who wear hearing aids have an increased risk of crashing compared to drivers who do not wear them (excluding individuals who formerly wore hearing aids then discarded them, who had an even worse driving record). Thus, auditory train signals may not be completely effective as a secondary warning system for visually impaired drivers or drivers who neglect to properly scan at rail crossings if they are also hearing impaired. At the same time, data show that audible warnings can help reduce nighttime crashes, as evidenced by the 195 percent increase in collisions in Florida as a result of a nighttime whistle ban between 10:00 p.m. and 6:00 a.m. (Kinnan, 1993). Raslear's (1995) crash prediction model indicates that the use of the train whistle reduces the field of visual search from 180 degrees to 10 degrees, which, in turn, reduces the visual search time by a factor of 18. By decreasing visual search time, the train whistle decreases the probability of a crash.

Though few studies have directly measured the effectiveness of countermeasures for older drivers in this arena, sufficient data exist to explain performance errors among the population at large to support highway-rail grade crossing design element recommendations for passive crossing control devices that offer the greatest promise to improve safety for older road users.

Recommendations by Design Element

Design Element: Passive Crossing Control Devices

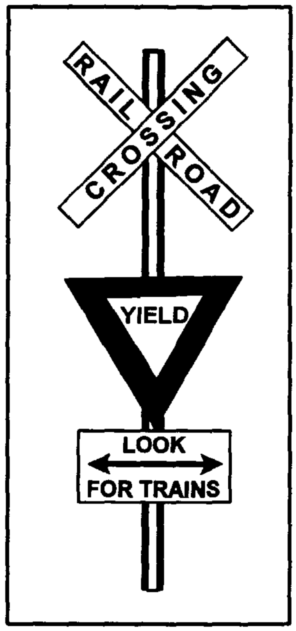
(1) To increase the conspicuity and comprehensibility of elements marking the location of the grade crossing under all operating conditions, it is recommended that:

MUTCD:4

(1a) The front and back of the crossbuck post be delineated (full length) with white high-brightness retroreflective sheeting with a minimum width of 50 mm (2 in).

MUTCD:4

(1b) A sign assembly, including a YIELD (R1-2) sign plus a supplemental panel below containing the legend LOOK FOR TRAINS and a bi-directional arrow, be added to the crossbuck post (as shown at right) in such a manner that the R1-2 sign is retroreflectorized with durable fluorescent sheeting, is no smaller than the *MUTCD* standard size (900 mm [36 in]), is mounted as close to 1200 mm (48 in) above the ground as is practical in a given location, and the supplemental panel consists of 100-mm (4-in) black letters on a white background.



RLH:4
RRX:4
MUTCD:4

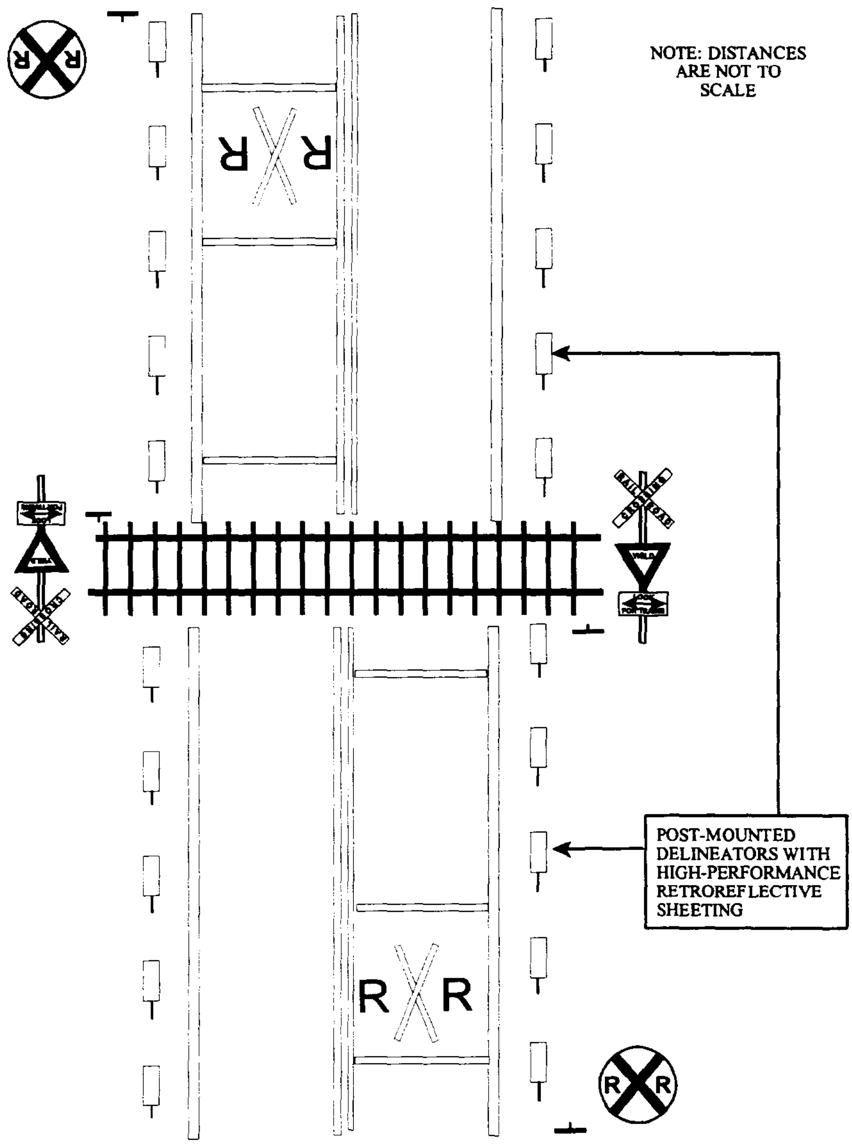
(2) Where crash experience or engineering study has determined a need for nighttime illumination as a safety countermeasure, it is recommended that full or semi-cutoff luminaires be used, aligned toward the track instead of toward the roadway geometry.

The rationale and supporting evidence for these recommendations can be found beginning on page 283 of this Handbook.

Design Element: Passive Crossing Control Devices (continued)

RRX:4

(3) For rural grade crossings that are not illuminated, it is recommended that the approach be delineated with post-mounted delineators spaced 15 m (50 ft) or closer together on the right shoulder, from the location of the Railroad Advance Warning sign (W10-1) to the crossbuck, and extending an equal distance beyond the crossbuck (as shown below).



The rationale and supporting evidence for this recommendation can be found beginning on page 283 of this Handbook.

RATIONALE AND SUPPORTING EVIDENCE

This section of the Handbook is organized in terms of the same classes of highway features as the recommendations: I. Intersections (At-Grade), II. Interchanges (Grade Separation), III. Roadway Curvature and Passing Zones, IV. Construction/Work Zones, and V. Highway-Rail Grade Crossings (Passive). Within each of these five classes, subsections are organized in terms of design elements with unique geometric, operational, and/or traffic control characteristics, also consistent with the recommendations.

At the beginning of each subsection within a class of highway features, reference material for a particular design element is introduced using a cross-reference table. This table relates the discussion in that subsection-as well as the associated recommendations presented earlier-to entries in standard reference manuals consulted by practitioners in this area. Principal among these reference manuals are the *Manual on Uniform Traffic Control Devices* (Federal Highway Administration [FHWA], 2000); the *Policy on Geometric Design of Highways and Streets* [the *Green Book*] (American Association of State Highway and Transportation Officials [AASHTO], 1994); and the *Traffic Engineering Handbook* (ITE, 1999). Other standard references with more restricted applicability, which also appear in the cross-reference tables for selected design elements, include the National Cooperative Highway Research Program (NCHRP) Report No. 279, *Intersection Channelization Design Guide* (Neuman, 1985); *Roundabouts: An Informational Guide* (FHWA, 2000); the *Roadway Lighting Handbook* (FHWA, 1978); the *Railroad-Highway Grade Crossing Handbook* (FHWA, 1986); and the *Highway Capacity Manual* (TRB, 1998).

Material in this part of the Handbook represents, to as great an extent as possible at the time of its development, the results of empirical work with older driver or pedestrian samples for investigations with the specific highway features of interest. Observational and controlled field studies were given precedence, together with laboratory simulations employing traffic stimuli and relevant situational cues. Crash data are cited as appropriate. In addition, some citations reference studies showing the effects of design changes, where the predicted impact on (older) driver performance is tied logically to the results of research on age differences in response capability.

I. INTERSECTIONS (AT-GRADE)

The following discussion presents the rationale and supporting evidence for Handbook recommendations pertaining to these 17 design elements (A–Q):

A. Intersecting Angle (Skew)
B. Receiving Lane (Throat) Width for Turning Operations
C. Channelization
D. Intersection Sight-Distance Requirements
E. Offset (Single) Left-Turn Lane Geometry, Signing, and Delineation
F. Treatments/Delineation of Edgelines, Curbs, Medians, and Obstacles
G. Curb Radius
H. Traffic Control for Left-Turn Movements at Signalized Intersections
I. Traffic Control for Right-Turn/RTOR Movements at Signalized Intersections
J. Street-Name Signing
K. One-Way/Wrong-Way Signing
L. Stop- and Yield-Controlled Intersection Signing
M. Devices for Lane Assignment on Intersection Approach
N. Traffic Signals
O. Fixed Lighting Installations
P. Pedestrian Crossing Design, Operations, and Control
Q. Roundabouts

A. Design Element: Intersecting Angle (Skew)

Table 1. Cross-references of related entries for intersecting angle (skew).

Applications in Standard Reference Manuals			
MUTCD (2000)	AASHTO *Green Book* (1994)	NCHRP 279 *Intersection Channelization Design Guide* (1985)	*Traffic Engineering Handbook* (1999)
Sects. 2B.39 & 4D.17	p. 426, Para. 5 p. 628, Item C.4 p. 630, Para. 1 pp. 641-645, Sects. on *Multileg Intersections & Alinement* pp. 648-651, Tables IX-1 & IX-2 pp. 663-664, Sect. on *Oblique-Angle Turns* p. 673, Para. 5 pp. 676-680, Sects. *on Divisional Islands, Refuge Islands, & Island Size and Designation* Fig. IX-23 pp. 689-690, Sect. on *Oblique-Angle Turns With Corner Islands* p. 691, Table IX-4 pp. 718-720, Sect. on *Effect of Skew* pp. 764-767, Sect. on *Effect of Skew*	p. 19, Top fig. p. 21, Item 5 p. 25, Para. 2 p. 30, Para. 1 & top three figs. p. 31, Para. 3 & bottom left fig. pp. 42-44, Sect. on *Angle of Intersection* p. 45, Figs. 4-5 p. 71, Top two figs. pp 100-105, Intersect. Nos. 7 -9 pp. 148-149, Intersect. No. 35	p. 384, 5th Principle p. 385, Sect. on *Angle of Intersection* p. 399, Para. 2 p. 435, Para. 4

There is broad agreement that right-angle intersections are the preferred design. Decreasing the angle of the intersection makes detection of and judgments about potential conflicting vehicles on crossing roadways much more difficult. In addition, the amount of time required to maneuver through the intersection increases, for both vehicles and pedestrians, due to the increased pavement area. However, there is some inconsistency among reference sources concerning the degree of skew that can be safely designed into an intersection. The *Green Book*

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states that although a right-angle crossing normally is desired, an angle of 60 degrees provides most of the benefits that are obtained with a right-angle intersection. Subsequently, factors to adjust intersection sight distances for skewness are suggested for use only when angles are less than 60 degrees (AASHTO, 1994). However, another source on subdivision street design states that: "Skewed intersections should be avoided, and in no case should the angle be less than 75 degrees" (Institute of Transportation Engineers [ITE], 1984). The *Traffic Engineering Handbook* (ITE, 1999) states that: "Crossing roadways should intersect at 90 degrees if possible, and not less than 75 degrees." It further states that: "Intersections with severe skew angles (e.g., 60 degrees or less) often experience operational or safety problems. Reconstruction of such locations or institution of more positive traffic control such as signalization is often necessary." With regard to intersection design issues on two-lane rural highways, ITE (1999) states that: "Skew angles in excess of 75 degrees often create special problems at stop-controlled rural intersections. The angle complicates the vision triangle for the stopped vehicle; increases the time to cross the through road; and results in a larger, more potentially confusing intersection."

Skewed intersections pose particular problems for older drivers. Many older drivers experience a decline in head and neck mobility, which accompanies advancing age and may contribute to the slowing of psychomotor responses. Joint flexibility, an essential component of driving skill, has been estimated to decline by approximately 25 percent in older adults due to arthritis, calcification of cartilage, and joint deterioration (Smith and Sethi, 1975). A restricted range of motion reduces an older driver's ability to effectively scan to the rear and sides of his or her vehicle to observe blind spots, and similarly may be expected to hinder the timely recognition of conflicts during turning and merging maneuvers at intersections (Ostrow, Shaffron, and McPherson, 1992). For older drivers, diminished physical capabilities may affect their performance at intersections designed with acute angles by requiring them to turn their heads farther than would be required at a right-angle intersection. This obviously creates more of a problem in determining appropriate gaps. For older pedestrians, the longer exposure time within the intersection becomes a major concern.

Isler, Parsonson, and Hansson (1997) measured the maximum head rotation and horizontal peripheral visual field of 20 drivers in each of four age groups: less than age 30; ages 40 to 59; ages 60 to 69; and age 70 and older. The oldest subjects exhibited an average decrement of approximately one-third for head range of movement compared with the youngest group of subjects. The mean maximum head movement (in one direction) was 86 degrees for the youngest drivers, 72 degrees for drivers ages 40 to 59, 67 degrees for drivers ages 60 to 69, and 59 degrees for drivers age 70 and older. In addition, the percentage of drivers with less than 30 degrees of horizontal peripheral vision increased with increased age, from 15 percent of the younger driver sample to 65 percent of the drivers age 70 and older. Three of the oldest drivers had less than 50 degrees of head movement and two of these drivers also had less than 20 degrees of horizontal peripheral vision.

In a survey of older drivers conducted by Yee (1985), 35 percent of the respondents reported problems with arthritis and 21 percent indicated difficulty in turning their heads to scan rearward while driving. Excluding vision/visibility problems associated with nighttime operations, difficulty with head turning placed first among *all* concerns mentioned by older drivers participating in a more recent focus group conducted to examine problems in the use of intersections where the approach leg meets the main road at a skewed angle, and/or where channelized right-turn lanes require an exaggerated degree of head/neck rotation to check for traffic conflicts before merging (Staplin, Harkey, Lococo, and Tarawneh, 1997). Comments about this geometry centered around the difficulty older drivers experience turning their heads at angles less than 90 degrees to view traffic on the intersecting roadway, and several participants reported an increasing reliance on outside rearview mirrors when negotiating highly skewed angles. However, they reported that the outside mirror is of no help when the roads meet at the middle angles (e.g., 40 to 55 degrees) and a driver is not flexible enough to physically turn to look for traffic. In an observational field study conducted as part of the same project, Staplin et al. (1997) found that approximately 30 percent of young/middle-aged drivers (ages 25–45) and young-old drivers (ages 65–74) used their mirrors in addition to making head checks before performing a right-turn-on-red (RTOR) maneuver at a skewed intersection (a channelized right-turn lane at a 65-degree skew). By comparison, none of the drivers age 75 and older used their mirrors; instead, they relied solely on information obtained from head/neck checks. In this same study, it was found that the likelihood of a driver making an RTOR maneuver is reduced by intersection skew angles that make it more difficult for the driver to view conflicting traffic.

The practical consequences of restricted head and neck movement on driving performance at T-intersections were investigated by Hunter-Zaworski (1990) using a simulator to present video-recorded scenes of intersections with various levels of traffic volume and sight distance in a 180-degree field of view from the driver's perspective. Drivers in two subject groups, ages 30–50 and 60–80, depressed a brake pedal to watch a video presentation (on three screens), then released the pedal when it was judged safe to make a left turn; half of *each* age group had a restricted range of neck movement as determined by goniometric measures of maximum (static) head-turn angle. Aside from demonstrating that skewed intersections are hazardous for any driver with an impairment in neck movement, this study found that maneuver decision time increased with both age *and* level of impairment. Thus, the younger drivers in this study were able to compensate for their impairments, but older drivers both with and without impairments were unable to make compensations in their (simulated) intersection response selections.

These research findings reinforce the desirability of providing a 90-degree intersection geometry and endorse the ITE (1984) recommendation establishing a 75-degree minimum as a practice to accommodate age-related performance deficits.

B. Design Element: Receiving Lane (Throat) Width for Turning Operations

Table 2. Cross-references of related entries for receiving lane (throat) width for turning operations.

Applications in Standard Reference Manuals		
AASHTO *Green Book* **(1994)**	**NCHRP 279,** ***Intersection Channelization Design Guide*** **(1985)**	***Traffic Engineering Handbook*** **(1999)**
pp. 200-211, Sects. on *Widths for Turning Roadways at Intersections & Widths Outside Traveled Way Edges* p. 213, Table III-21 p. 647, Para. 2 p. 673, Para. 5 p. 676, Paras. 3-5 p. 678, Fig. X-24	p. 10, Table 2-4 p. 57, Para. 5, 1st Bullet p. 58, Fig. 4-20 p. 63, Sect. on *Lane Widths* p. 69, Sect. on *Width of Roadways* p. 73, Fig. 4-29 p. 107, Fig. c p. 113, Fig. a p. 115, Figs. d-e p. 120, Item 3 p. 122, Item 2 p. 125, Intersect. No. 19	p. 319, Para. 4 p. 386, Para. 5 p. 435, Para. 4

Design recommendations for lane width at intersections follow from consideration of vehicle maneuver requirements and their demands on drivers. Positioning a vehicle within the lane in preparation for turning has been rated as a critical task (McKnight and Adams, 1970). Swinging too wide to lengthen the turning radius and minimize rotation of the steering wheel ("buttonhook turn") while turning left or right is a common practice of drivers lacking strength (including older drivers) and physically limited drivers (McKnight and Stewart, 1990).

Two factors can compromise the ability of older drivers to remain within the boundaries of their assigned lanes during a left turn. One factor is the diminishing ability to share attention (i.e., to assimilate and concurrently process multiple sources of information from the driving environment). The other factor involves the ability to turn the steering wheel sharply enough, given the speed at which they are traveling, to remain within the boundaries of their lanes. Some older drivers seek to increase their turning radii by initiating the turn early and rounding-off the turn. The result is either to cut across the apex of the turn, conflicting with vehicles approaching from the left, or to intrude upon a far lane in completing the turn.

Lane widths are addressed in the *Intersection Channelization Design Guide* (Neuman, 1985). A recommendation for (left) turning lanes, which also applies to receiving lanes, is that "3.6-m (12-ft) widths are desirable, (although) lesser widths may function effectively and safely. Absolute minimum widths of 2.7 m (9 ft) should be used only in unusual circumstances, and only on low-speed streets with minor truck volumes." Similarly, the ITE (1984) guidelines suggest a minimum lane width of 3.3 m (11 ft) and specify 3.6 m (12 ft) as desirable. These guidelines suggest that wider lanes be avoided due to the resulting increase in pedestrian crossing distances. However, the ITE guidelines provide a range of lane widths at intersections from 2.7 m to 4.3 m (9 ft to 14 ft), where the wider lanes would be used to accommodate larger turning vehicles, which have turning paths that sweep a path from 4.1 m (13.6 ft) for a single-unit truck or bus, up to 6.3 m (20.6 ft) for a semitrailer. Thus, wider (3.6 m [12 ft]) lanes used to accommodate (right) turning trucks also are expected to benefit (left) turning drivers. However, further increases in lane width for accommodation of heavy vehicles may result in unacceptable increases in (older) pedestrian crossing times.

Results of field observation studies conducted by Firestine, Hughes, and Natelson (1989) found that trucks performing turns on urban roads encroached into other lanes on streets with widths of less than 3.6 m (12 ft). They noted that on rural roads, lanes wider than 3.6 m or 4.0 m (12 ft or 13 ft) allowed oncoming vehicles on the cross street to move farther right to avoid trucks, and shoulders wider than 1.2 m (4 ft) allowed oncoming vehicles a greater margin of safety.

In an observational field study conducted to determine how older drivers (age 65 and older) compare with younger drivers during left-turn operations under varying intersection geometries, one variable that showed significant differences in older and younger driver behavior was turning path (Staplin, Harkey, Lococo, and Tarawneh, 1997). Older drivers encroached into the opposing lane of the cross-street (see figure 1, turning-path trajectory no. 1) when making the left turn more often than younger drivers at the location where the throat width (equivalent to the lane width) measured 3.6 m (12 ft). Where the throat width measured 7 m (23 ft), which consisted of a 3.6-m (12-ft) lane and a 3.3-m (11-ft) shoulder, there was no significant difference in the turning paths. The narrower throat width resulted in higher encroachments by older drivers, who physically may have more difficulties maneuvering their vehicles through smaller areas.

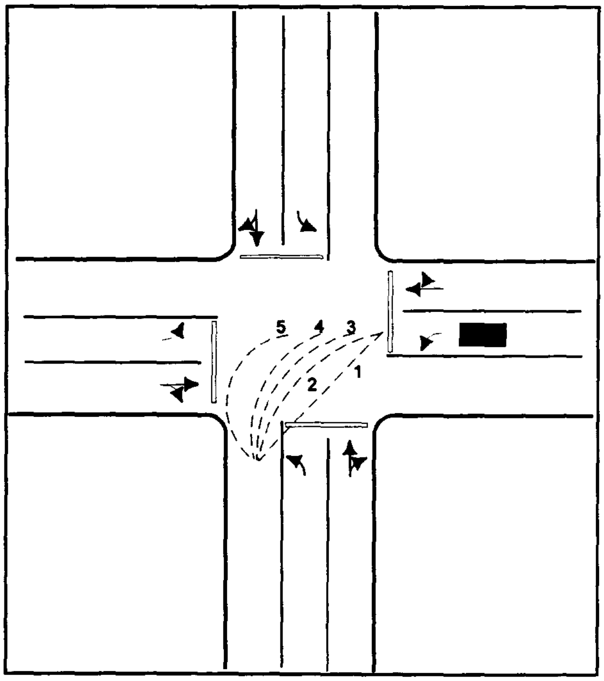


Figure 1. Turning path taken by left-turning vehicles, where 1=encroach into opposing cross-traffic stream; 2, 3, and 4=proper turning from different points within the intersection; and 5=left turn from a position requiring a greater-than-90-degree turn to enter the cross-street.

These data sources indicate that a 3.6-m (12-ft) lane width provides the most reasonable tradeoff between the need to accommodate older drivers, as well as larger turning vehicles, and avoiding penalizing the older pedestrian in terms of exaggerated crossing distance.

C. Design Element: Channelization

Table 3. Cross-references of related entries for channelization.

Applications in Standard Reference Manuals				
MUTCD (2000)	AASHTO *Green Book* (1994)	*Roadway Lighting Handbook* (1978)	NCHRP 279, *Intersection Channelization Design Guide* (1985)	*Traffic Engineering Handbook* (1999)
Sect. 1A.13, *channelizing line markings* Sect. 3B.03 Sect. 3B.05 Sects. 3B.09, 3B.10, 3B.19, 3B.21, 3E.01, 3F.02, 3G.01 through 3G.06, & 5G.03	p. 369, Para. 2 p. 517, Paras. 5-6 p. 518, Fig. VII-8 pp. 631-632, Sect. on *Channelized Three-Leg Intersections* pp. 635-641, Sect. on *Channelized Four-Leg Intersections* pp. 674-689, Sects. on *Channelized Islands, Divisional Islands, Refuge Islands, Island Size and Designation, Delineation and Approach-End Treatment, & Right-Angle Turns With Corner Island* pp. 740-749, Sects. on *General Design Considerations & Channelization* p. 778, Sect. on *Continuous Left-Turn Lanes (Two-Way)*	p. 2, 2nd col. Para. 1 p. 3. Para. 3 p. 18, Form 2 p. 21, Table 1 p. 22, Table 2 p. 26. 3rd col. Para. 2 p. 71, 5th bullet p. 99, Para. 3	p. 1, Paras. 2-3 p. 21, Fig. 3-1 p. 24, Bottom fig. p. 25, Para. 3 p. 26, Top fig. p. 28, Middle fig. p. 32, Middle fig. p. 34, Para. 1 & bottom fig. p. 35, Bottom left fig. p. 38, Middle fig. p. 39, Paras. 2-3 & top two figs. p. 69, Sect. on *Traffic Islands* p. 74, Fig. 4-30 pp.75-76, Para. 1 on 1st page & Sects. on *Guidelines for Design of Traffic Islands, Guidelines for Selection of Island Type, & Guidelines for Design of Median Islands* p. 79, Fig. 4-34 pp. 94- 95, Intersct. No. 4 pp. 102-103, Intersct. No. 8 pp. 106-113, Intersct. Nos. 10-13 pp. 116-117, Intersct. No. 15 pp. 132-133, Intersct. No. 22 pp. 138-139, Intersct. No. 29 pp. 148-153, Intersct. Nos. 35-37	p. 319, Para. 4 pp. 384-385, Sect. on *Principles of Intersection Channelization* p. 388, Sect. on *Traffic Island Design* p. 405, Para. 4 p. 434, Sect. on *Channelizing Lines* p. 435, Para. 4 p. 438, Item 5 p. 439, Para. 5 p. 440, Sect. on *Channelization* pp. 441-443, Sect. on *Channelizing Traffic Control Devices*

The spatial visual functions of acuity and contrast sensitivity are important in the ability to detect/recognize downstream geometric features such as pavement width transitions, channelized turning lanes, island and median features across the intersection, and any non-reflectorized raised elements at intersections. Visual acuity (the ability to see high-contrast, high-spatial-frequency stimuli, such as black letters on a white eye chart) shows a slow decline beginning at approximately age 40, and marked acceleration at age 60 (Richards, 1972). Approximately 10 percent of men and women between ages 65 and 75 have (best corrected) visual acuity worse than 20/30, compared with roughly 30 percent over age 75 (Kahn, Leibowitz, Ganley, Kini, Colton, Nickerson, and Dawber, 1977). A driver's response to intersection geometric features is influenced in part by the processing of high-spatial-frequency cues-for example, the characters on upstream advisory signs-but it is the larger, often diffuse edges defining lane and pavement boundaries, curb lines, and raised median barriers that are the targets with the highest priority of detection for safety. Older persons' sensitivity to visual contrast (the ability to see objects of various shapes and sizes under varying levels of contrast) also declines beginning around age 40, then declines steadily as age increases (Owsley, Sekuler, and Siemsen, 1983). Poor contrast sensitivity has been shown to relate to increased crash involvement for drivers age 66 and older, when incorporated into a battery of vision tests also including visual acuity and horizontal visual field size (Decina and Staplin, 1993).

The effectiveness of channelization from a safety perspective has been documented in several studies. An evaluation of Highway Safety Improvement Program projects showed that channelization produced an average benefit-cost ratio of 4.5 (FHWA, 1996). In this evaluation, roadway improvements consisting of turning lanes and traffic channelization resulted in a 47 percent reduction in fatal crashes, a 26 percent reduction in non-fatal injury crashes, and a 27 percent reduction in combined fatal plus non-fatal injury crashes, at locations where before and after exposure data were available.

One of the advantages of using curbed medians and intersection channelization is that it provides a better indication to motorists of the proper use of travel lanes at intersections. In a set of studies performed by the California Department of Public Works investigating the differences in crash experience with raised channelization versus channelization accomplished through the use of flush pavement markings, the findings were as follows: raised traffic islands are more effective than flush marked islands in reducing the frequency of nighttime crashes, particularly in urban areas; and little difference is noted in the effectiveness of raised versus marked channelizing islands at rural intersections (Neuman, 1985).

One of the most common uses of channelization is for the separation of left-turning vehicles from the through-traffic stream. The safety benefits of left-turn channelization have been documented in several studies. A study by McFarland, Griffin, Rollins, Stockton, Phillips, and Dudek (1979) showed that crashes at signalized intersections where a left-turn lane was added, in combination with and without a left-turn signal phase, were reduced by 36 percent and 15 percent, respectively. At non-signalized intersections with marked channelization separating the left-turn lane from the through lane, crashes were reduced for rural, suburban, and urban areas by 50, 30, and 15 percent, respectively. When raised channelization devices were used, the crash reductions were 60, 65, and 70 percent in rural, suburban, and urban areas, respectively. Consistent findings were reported in Hagenauer, Upchurch, Warren, and Rosenbaum (1982).

Important considerations in choosing to implement raised versus marked channelization include operating speed and type of maneuver (i.e., left turn versus right turn). Left-turn channelization separating through and turning lanes may, because of its placement, constitute a hazard when a raised treatment is applied, especially on high-speed facilities. Detection and avoidance of such hazards requires visual and response capabilities known to decline significantly with advancing age.

In this same vein, it was reported in *Transportation Research Circular 382* (Transportation Research Board, 1991) that the aging driver, having poorer vision, slower physical reaction time, lower degree of awareness, and reduced ability to maneuver the vehicle, is more likely to be *negatively* affected by a raised median than is the average driver; and, because medians are fixed objects, when they are struck they pose a serious threat of loss of control, especially for aging drivers. The typical curbed median offers low to no contrast with the adjacent pavement and is difficult to reflectorize at night. Low-beam headlight limitations, coupled with the reduced vision of the aging driver, compounds the visibility problem. In addition, raised medians and raised corner islands, when used together, often create turning-path options at complex intersections that are confusing to the average driver, and disproportionately so for the aging one. Thus, to realize the safety benefits channelization can provide, it is particularly important to ensure the visibility of raised surfaces for (older) drivers with diminished vision, so these road users can detect the channelizing devices and select their paths accordingly.

Another benefit in the use of channelization is the provision of a refuge for pedestrians. Refuge islands are a design element that can aid older pedestrians who have slow walking speeds. With respect to the Hagenauer et al. (1982) study cited earlier, Hauer (1988) stated that because channelization in general serves to simplify an otherwise ambiguous and complex situation, the channelization of an existing intersection might enhance both the safety and mobility of older persons, as well as enhance the safety of other pedestrians and drivers. However, in designing a new intersection, he stated that the presence of islands is unlikely to offset the disadvantage of large intersection size for the pedestrian.

Staplin, Harkey, Lococo, and Tarawneh (1997) conducted a field study evaluating four right-turn lane geometries to examine the effect of channelized right-turn lanes and the presence of skew on right-turn maneuvers made by drivers of different ages. One hundred subjects divided across three age groups drove their own vehicles around test routes using the local street network in Arlington, VA. The three age groups were young/middle-aged (ages 25–45), young-old (ages 65–74), and old-old (age 75 and older). As diagrammed in figure 2, the four right-turn lane geometries were:

(a) A non-channelized 90-degree intersection where drivers had the chance to make a right turn on red (RTOR) around a 12.2-m (40-ft) radius. This site served as a control geometry to examine how channelized intersections compare with non-channelized intersections.

(b) A channelized right-turn lane at a 90-degree intersection with an exclusive-use (acceleration) lane on the receiving street. Under this geometric configuration, drivers did not need to stop at the intersection and they were removed from the conflicting traffic upon entering the cross-street. They had the opportunity to accelerate in their own lane on the cross-street and then change lanes downstream when they perceived that it was safe to do so.

(c) A channelized right-turn lane at a 65-degree skewed intersection without an exclusive-use lane on the receiving street.

(d) A channelized right-turn lane at a 90-degree intersection without an exclusive-use lane on the receiving street. Under this geometry, drivers needed to check the conflicting traffic and complete their turn into a through-traffic lane on the cross-street.

The right-turn maneuver at all locations was made against two lanes carrying through (conflicting) traffic. The two through lanes were the only ones that had a direct effect on the

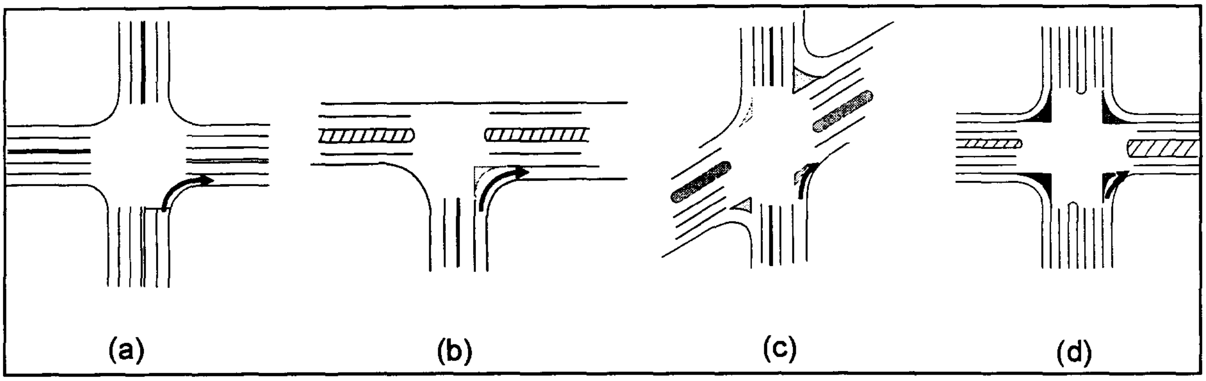


Figure 2. Intersection geometries examined in the Staplin et al. (1997) field study of right-turn channelization.

right-turn maneuver. All intersections were located on major or minor arterials within a growing urban area, where the posted speed limit was 56 km/h (35 mi/h). All intersections were controlled by traffic signals with yield control on the three channelized intersections.

The results indicated that right-turn channelization affects the speed at which drivers make right turns and the likelihood that they will stop before making an RTOR. Drivers, especially younger drivers (ages 25–45), turned right at speeds 4.8–8 km/h (3–5 mi/h) higher on intersection approaches with channelized right-turn lanes than they did on approaches with non-channelized right-turn lanes.

At the non-channelized intersection, 22 percent of the young/middle-aged drivers, 5 percent of the young-old drivers, and none of the old-old drivers performed an RTOR without a stop. On approaches with channelized right-turn lanes, young/middle-aged and young-old drivers were much less likely to stop before making an RTOR. Where an acceleration lane was available, 65 percent of the young/middle-aged drivers continued through without a complete stop, compared with 55 percent of the young-old drivers and 11 percent of the old-old drivers. Old-old female drivers *always* stopped before an RTOR. The increased mobility exhibited by the two younger groups of drivers at the channelized right-turn lane locations was not, however, exhibited by the old-old drivers (age 75 and older), who stopped in 19 of the 20 turns executed at the channelized locations. Also, questionnaire results indicated that drivers perceived that making a right turn on an approach with a channelized right-turn lane *without an acceleration lane on the cross-street* was more difficult than at other locations, and even more difficult than at skewed intersections.

Regarding channelization for mid-block left-turn treatments, Bonneson and McCoy (1997) evaluated the safety and operational effects of three mid-block left-turn treatments: (1) raised-curb medians; (2) two-way, left turn-lanes; and (3) undivided cross-sections. Traffic flow data were collected during 32 field studies in 8 cities in 4 States, and 3-year crash histories for 189 street segments were obtained from cities in 2 States. The studies were conducted on urban or suburban arterial segments, and therefore recommendations can only be applied to such environments that include the following criteria: traffic volume exceeding 7,000 vehicles per day; speed limit between 48 and 80 km/h (30 and 50 mi/h); spacing of at least 107 m (350 ft) between signalized intersections; direct access from abutting properties; no angle curb parking (parallel parking is acceptable); located in or near a populated area (e.g., population of 20,000 or more); no more than six through lanes (three in each direction); and an arterial length of at least 1.2 km (0.75 mi).

In terms of annual delays to major-street left-turn and through vehicles, the raised-curb treatment has slightly higher delays than the two-way, left-turn lane (TWLTL) treatment at the highest left-turn and through volumes, which results from the greater likelihood of bay overflow for the raised-curb median treatment under high-volume conditions. The undivided cross-section has significantly higher delays than the raised-curb treatment for all non-zero combinations of left-turn and through volume.

Looking at crash frequencies as a function of midblock channelization treatment, the raised-curb median treatment is associated with the fewest crashes of all three treatment types. Differences between the crash frequencies for TWLTL treatments versus undivided cross-sections are affected by whether or not parallel parking is allowed on the undivided cross-section. When parallel parking is allowed on the undivided cross-section, the undivided cross-section is associated with significantly more crashes than the TWLTL treatment. However, when parallel parking is not allowed, the TWLTL has about the same crash frequency as the undivided cross-section at lower traffic volumes.

In general, at midblock locations, the raised-curb median treatment was associated with fewer crashes than the undivided cross-section and TWLTL, especially for average daily traffic demands greater than 20,000 vehicles per day. Also, a benefit of the raised-curb median is that it provides a pedestrian refuge.

Bonneson and McCoy (1997) provide a set of six tables to use as guidelines in considering the conversion of an undivided cross-section to a raised-curb median, or to a TWLTL, and conversion from a TWLTL to a raised-curb median. In these tables, it is recommended that the existing treatment remain in place when the benefit-cost ratio (in terms of delay and safety) is less than 1.0, and when the benefit-cost ratio exceeds 2.0, it is recommended that the engineer consider adding the alternative treatment.

Bonneson and McCoy (1997) do not report crash frequencies by driver age for one treatment versus another. However, approximately one-fifth of the older drivers participating in focus group studies conducted by Staplin, Harkey, Lococo, and Tarawneh (1997) reported that using a center TWLTL was confusing, risky, and made them uncomfortable because, at times, they have come face-to-face with an opposing left-turner, and both drivers were stranded. Also mentioned was the difficulty seeing the pavement markings in poor weather (night, fog, rain) when they are less visible, and particularly when they are snow-covered. Drivers referred to TWLTLs as "suicide lanes." In the same research study, Staplin et al. (1997) reported on a crash analysis that revealed ways in which older drivers failed to use a TWLTL correctly: (1) a TWLTL was not used for turning at all and (2) the TWLTL was entered too far in advance of where the turn was to be made.

D. Design Element: Intersection Sight-Distance Requirements

Table 4. Cross-references of related entries for intersection sight-distance requirements.

Applications in Standard Reference Manuals			
AASHTO *Green Book* (1994)	***Roadway Lighting Handbook* (1978)**	**NCHRP 279, *Intersection Channelization Design Guide* (1985)**	***Traffic Engineering Handbook* (1999)**
pp. 126-127, Sect. on *Decision Sight Distance* p. 440, Para. 5 p. 469, Para. 2 p. 491, Para. 1 p. 643, Para. 2 p. 645, Para. 1 pp. 646-647, Sect. on *Profile* pp. 696-724, Sects. on *Sight Distance & Stopping Sight Distance at Intersections for Turning Roadways* p. 796, Para. 5 through p. 801 pp. 938-939, Sects. on *Terminal Location and Sight Distance, Ramp Terminal Design, & Distance Between a Free-Flow Terminal and Structure*	p. 18, Form 2 p. 22, Table 2 p. 25, Table 3 Example	p. 1, Item, 1st bullet p. 10, Table 2-4 pp. 13-14, Sect. on *Sight Distance* p. 15, Para. 1 p. 27, Bottom right fig. p. 30, 2nd fig. from bottom p. 31, Para. 3 p. 35, Para. 3 & bottom right fig. p. 44, Para. 6, item 1 p. 45, Table 4-2 p. 63, Para. 3, item 3 p. 75, Last item 4 pp. 99-103, Intersect. Nos. 6-8 pp. 106-111, Intersect. Nos. 10-12	p. 238, Sect. on *Intersection Sight Distance* p. 339, Para. 3 pp. 375-376, Sect. on *Inter-section Sight Distance (ISD)* p. 405, Para. 4

Because at-grade intersections define locations with the highest probability of conflict between vehicles, adequate sight distance is particularly important. Not surprisingly, a number of studies have shown that sight-distance problems at intersections usually result in a higher crash rate (Mitchell, 1972; Hanna, Flynn, and Tyler, 1976; David and Norman, 1979). The need for adequate sight distance at an intersection is best illustrated by a quote from the *Green Book*: "The operator of a vehicle approaching an intersection at-grade should have an unobstructed view of the entire intersection and sufficient lengths of the intersecting highway to permit control of the vehicle to avoid collisions" (AASHTO, 1994). AASHTO values (for both uncontrolled and stop-controlled intersections) for available sight distance are measured from the driver's eye height (currently 1070 mm [3.25 ft]) to the roofline of the conflicting vehicle (currently 1300 mm [4.25 ft]).

Sight distances at an intersection can be reduced by a number of deficiencies, including physical obstructions too close to the intersection, severe grades, and poor horizontal alignment. The alignment and profile of an intersection have an impact on the sight distance available to the driver and thus affect the ability of the driver to perceive the actions taking place both at the intersection and on its approaches. Since proper perception is the first key to performing a safe maneuver at an intersection, it follows that sight distance should be maximized; this, in turn, means that the horizontal alignment should be straight and the gradients as flat as practical. Horizontal curvature on the approaches to an intersection makes it difficult for drivers to determine appropriate travel paths, because their visual focus is directed along lines tangential to these paths. Kihlberg and Tharp (1968) showed that crash rates increased 35 percent for highway segments with curved intersections versus highway segments with straight intersections. Limits for the vertical alignment at intersections that were suggested by AASHTO (1994) and ITE (1984) are 3 percent and 2 percent, respectively.

Harwood, Mason, Pietrucha, Brydia, Hostetter, and Gittings (1993) stated that the provision of intersection sight distance (ISD) is intended to give drivers an opportunity to obtain the information they need to make decisions about whether to proceed, slow, or stop in situations where potentially conflicting vehicles may be present. They noted that while it is desirable to provide a reasonable margin of safety to accommodate incorrect or delayed driver decisions, there are substantial costs associated with providing sight distances at intersections; therefore, it is important to understand the derivation of ISD requirements and why it is reasonable to expect a safety benefit from tailoring this design parameter to the needs of older drivers.

Traditionally, the need for-as well as the basis for calculating-sight distances at intersections has rested upon the notion of the *sight triangle*. This is diagrammed in figure 3. As excerpted from NCHRP Report 383, this diagram effectively illustrates how different driver decisions during a (minor) road approach to an intersection (with a major road) depend on the planned action. The driver's first decision is to either stop or to continue through the intersection (with a turning or a crossing maneuver) according to the type of traffic control information he or she perceives. A red signal or a stop sign results in a "stop" decision; all other types of information are functionally equivalent at this stage of driver decision making, translating into a "yield" decision. That is, drivers' decisions at this stage are dichotomous: (1) slow down and prepare to stop, regardless of traffic on the major road, or (2) based on their view of the major road, either slow down, maintain speed, or accelerate as required to safely complete their intended maneuver. For drivers who are required to stop, their decision to proceed after the stop also is based on a view of traffic on the major road, but at a point much closer to the intersection. The

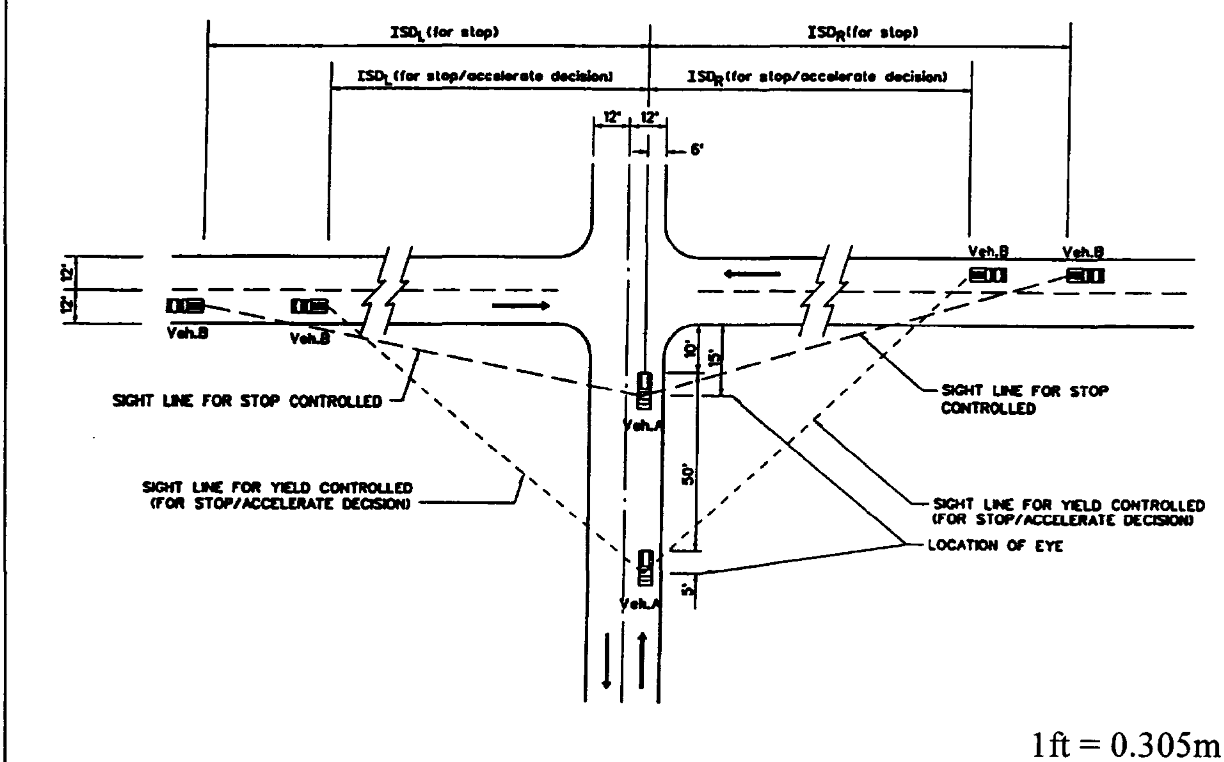


Figure 3. Sight distance for left and right turns for passenger car drivers at yield-controlled intersections. Source: Harwood et al. (1993).

contrasting sightlines and sight triangles defined by the position of a driver who *must* stop before proceeding at the intersection versus one who *may* proceed without stopping, conditional upon the intersecting (major) road traffic, are clearly indicated in figure 3.

For the purposes of describing driver decision making, figure 3 may apply to varying aspects of intersection operations in all Cases I through IV as *per* current AASHTO classification. For Case V, however, where a driver is turning left from a major road at an intersection or driveway, the decision-making process and corresponding sight-distance requirements are defined differently. The sightlines in this case are defined by the presence, type (passenger versus heavy vehicle), and location (positioned or unpositioned in the intersection) of opposing left-turning traffic, and by the lateral offset of the opposite left-turn lanes themselves. These relationships are illustrated in figure 4, taken from McCoy, Navarro, and Witt (1992).

It is also important to acknowledge recent thinking (see Harwood et al., 1993) that bases

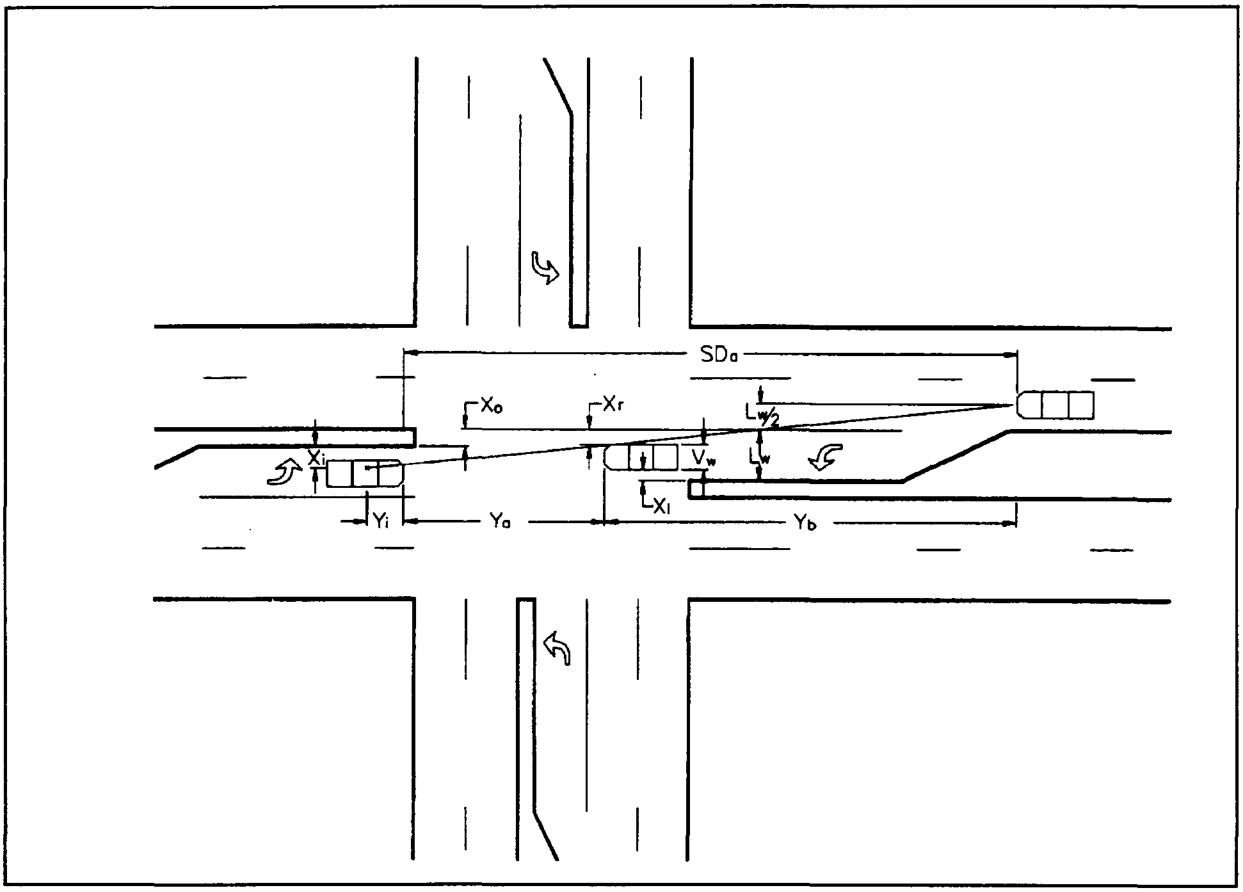


Figure 4. Spatial relationships that determine available sight distance.
Source: McCoy, Navarro, and Witt (1992).

recommended sight distances on the observed gaps that drivers will accept for performing various maneuvers at intersections-specifically, on the "critical gap." This is the distance, expressed in the number of seconds (at operating speed) of separation between a subject vehicle and a conflict vehicle, where the driver of the subject vehicle will make a decision to proceed with a maneuver *ahead of* the conflict vehicle 50 percent of the time. The Gap Acceptance model yields different, typically shorter, ISD requirements than the existing (or modified) AASHTO model. It also classifies intersection operations in a somewhat different manner than AASHTO (1994).

Both the assumptions underlying the current AASHTO (1994) model and the Gap Acceptance model, respectively, received consideration in developing the Handbook's recommendations for this design element. Apart from the theoretical differences between these models, there is also the practical matter that it may take some time for designers and engineers who are familiar with and have worked successfully with one approach to embrace an alternative approach. Thus, this Handbook seeks to accommodate the full range of design practices to the extent that data provide an understanding of the ISD requirements of older drivers. Where ISD requirements are defined through application of formulas incorporating perception-reaction time (PRT), the broad and well-documented age differences in this aspect of driver performance support recommendations for all included cases (I through V). Where ISD requirements are determined through a formula that depends on gap size, however, recommendations must be limited at this time to cases where gap acceptance by older versus younger drivers has been empirically studied (Case F).

The rationale for recommendations pertaining to ISD requirements will proceed as follows: First, driver age differences in cognitive and physical capabilities that are relevant to ISD issues will be discussed. Then, research efforts that have attempted to quantify the safety impact of providing adequate sight distance are summarized, as well as studies examining the appropriate values for specific components used when calculating sight distance with the AASHTO and Gap Acceptance models.

Older road users do not necessarily react more slowly to events that are expected, but they take significantly longer to make decisions about the appropriate response than younger road users, and this difference becomes more exaggerated in complex situations. Although the cognitive aspects of safe intersection negotiation depend on a host of specific functional capabilities, the net result is the slowing of response. There is general consensus among investigators that older adults tend to process information more slowly than younger adults, and that this slowing not only transcends the slower reaction times often observed in older adults, but may, in part, explain them (Anders, Fozard, and Lillyquist, 1972; Eriksen, Hamlin, and Daye, 1973; Waugh, Thomas, and Fozard, 1978; Salthouse and Somberg, 1982; Byrd, 1984). Of course, a conflict must be *seen* before any cognitive processing of this sort proceeds. Therefore, any decrease in available response time because of sight-distance restrictions will pose disproportionate risks to older drivers. Slower reaction times for older versus younger adults when response uncertainty is increased have been demonstrated by Simon and Pouraghabagher (1978), indicating a disproportionately heightened degree of risk when older road users are faced with two or more choices of action. Also, research has shown that older persons have greater difficulty in situations where planned actions must be rapidly altered (Stelmach, Goggin, and Amrhein, 1988). The difficulty older persons experience in making extensive and repeated head movements further increases the decision and response times of older drivers at intersections.

David and Norman (1979) quantified the relationship between available sight distance and the expected reduction in crashes at intersections. The results of this study showed that intersections with shorter sight distances generally have higher crash rates. Using these results, predicted crash reduction frequencies related to ISD were derived as shown in table 5.

Table 5. Expected reduction in number of crashes per intersection per year. Source: David and Norman (1979).

AADT* **(1000s)**	**Increased Sight Distance (ft)**		
	20-49	**50-99**	**>100**
< 5	0.18	0.20	0.30
5 - 10	1.00	1.30	1.40
10 - 15	0.87	2.26	3.46
> 15	5.25	7.41	11.26

1 ft = 0.305m

* *Annual average daily traffic entering the intersection.*

Other studies have attempted to show the benefits to be gained from improvements to ISD (Mitchell, 1972; Strate, 1980). Mitchell conducted a before-and-after analysis, with a period of 1 year on each end, of intersections where a variety of improvements were implemented. The results showed a 67 percent reduction (from 39 to 13) in crashes where obstructions that inhibited sight distance were removed; this was the most effective of the implemented improvements. Strate's analysis examined 34 types of improvements made in FHWA Safety Program projects. The results indicated that sight-distance improvements were the most cost-effective, producing a benefit-cost ratio of 5.33. The more recent report on the FHWA Highway Safety Improvement Programs (1996) indicates that improvements in ISD have a benefit-cost ratio of 6.1 in reducing fatal and injury crashes. In these analyses, fatal crashes were reduced by 56 percent and non-fatal injury crashes by 37 percent after sight-distance improvements were implemented.

Collectively, the studies described above indicate a positive relationship between available ISD and a reduction in crashes, though the amount of crash reduction that can be expected by a given increase in sight distance may be expected to vary according to the maneuver scenario and existing traffic control at the intersection. Procedures for determining appropriate ISDs are provided by AASHTO for various levels of intersection control and the maneuvers to be performed. The scenarios defined are as follows:

- Case I: No Control. ISD for vehicles approaching intersections with no control, at which vehicles are not required to stop, but may be required to adjust speed.

- Case II: Yield Control. ISD for vehicles on a minor-road approach controlled by a yield sign.

- Case IIIA: Stop Control-Crossing Maneuver. ISD for a vehicle on a stop-controlled approach on the minor road to accelerate from a stopped position and cross the major road.

- Case IIIB: Stop Control-Left Turn. ISD for a vehicle on a stop-controlled approach on the minor road to accelerate from a stopped position and turn left onto the major road.

- Case IIIC: Stop Control-Right Turn. ISD for a vehicle on a stop-controlled approach on the minor road to accelerate from a stopped position and turn right onto the major road.

- Case IV: Signal Control (should be designed by Case III conditions). ISD for a vehicle on a signal-controlled approach.

- Case V: Stop Control-Vehicle Turning Left From Major Highway Onto a Minor Roadway. ISD for a vehicle stopped on a major road, waiting to turn left across opposing lanes of travel.

By comparison, for applications of the Gap Acceptance model, an alternative classification system has been proposed (Harwood et al., 1996):

- Case A: Intersections with no control.

- Case B: Intersections with stop control on the minor road.
  - Case B1: Left turn from the minor road.
  - Case B2: Right turn from the minor road.
  - Case B3: Crossing maneuver from the minor road.

- Case C: Intersections with yield control on the minor road.
  - Case C1: Crossing maneuver from the minor road.
  - Case C2: Left or right turn from the minor road.

- Case D: Intersections with traffic signal control.

- Case E: Intersections with all-way stop control.

- Case F: Intersections where a stopped vehicle is turning left from a major road.

One of the principal components in determining ISD in all cases defined according to AASHTO (1994) is perception-reaction time (PRT). The discussion of this value is first presented in chapters 2 and 3 of the *Green Book* under "Reaction Time" and "Brake Reaction Time," respectively (AASHTO, 1994). Results of several studies (e.g., Normann, 1953; Johansson and Rumar, 1971) are cited, and in conclusion, the 2.5-s value is selected since it was found to be adequate for approximately 90 percent of the overall driver population. Controlled field studies and simulator studies involving older drivers have confirmed that brake reaction times to unexpected hazards (e.g., a barrel rolling into the road in front of the driver; a vehicle turning in front of a driver who is traveling straight through an intersection) are not significantly different as a function of age, and that virtually all response times are captured by the current 2.5-s AASHTO design parameter for brake PRT (Lerner, Huey, McGee, and Sullivan, 1995; Kloeppel, Peters, James, Fox, and Alicandri, 1995).

With respect to at-grade intersections, AASHTO recommends the following values of PRT for ISD calculations. In Case I, the PRT is assumed to be 2.0 s plus an additional 1.0 s to actuate braking, although the "preferred design" uses stopping sight distance (SSD) as the ISD design value (which incorporates a PRT of 2.5 s). In Case II, SSD is the design value; thus, the PRT is 2.5 s. For all Case III scenarios and Cases IV and V, the PRT is assumed to be 2.0 s.

Regarding PRT for Cases III and V, the value of 2.0 s assumed by AASHTO (1994) represents the time necessary for the driver to look in both directions of the roadway; to perceive that there is sufficient time to perform the maneuver safely; and to shift gears, if necessary, prior to starting. This value is based on research performed by Johansson and Rumar (1971). The PRT is defined as the time from the driver's first look for possible oncoming traffic to the instant the car begins to move. Some of these operations are done simultaneously by many drivers, and some operations, such as shifting gears, may be done before searching for intersecting traffic or may not be required with automatic transmissions. AASHTO states that a value of 2.0 s is assumed to represent the time taken by the slower driver.

A critique of these values questioned the basis for reducing the PRT from the 2.5 s used in SSD calculations to 2.0 s in the Case III ISD calculations (Alexander, 1989). As noted by the author, "The elements of PRT are: detection, recognition, decision, and action initiation." For SSD, this is the time from object or hazard detection to initiation of the braking maneuver. Time to search for a hazard or object is not included in the SSD computation, and the corresponding PRT value is 2.5 s. Yet, in all Case III scenarios, the PRT has been reduced to 2.0 s and now includes a search component that was not included in the SSD computations. Alexander pointed out that a driver is looking straight ahead when deciding to perform a stopping maneuver and only has to consider what is in his/her forward view. At an intersection, however, the driver must look forward, to the right, and to the left. This obviously takes time, especially for those drivers with lower levels of physical dexterity, e.g., older drivers. Alexander (1989) proposed the addition of a "search time" variable to the current equations for determining ISD, and use of the PRT value currently employed in the SSD computations (i.e., 2.5 s) for all ISD computations. Neuman (1989) also argued that a PRT of 2.5 s for SSD may not be sufficient in all situations, and can vary from 1.5 s to 5.0 s, depending on the physical state of the driver (alert versus fatigued), the complexity of the driving task, and the location and functional class of the highway.

A number of research efforts have been conducted to determine appropriate PRT values for use in ISD computations. Hostetter, McGee, Crowley, Seguin, and Dauber (1986) examined the PRT of 124 subjects traversing a 3-hour test circuit that contained scenarios identified above as Cases II, IIIA, IIIB, and IIIC. For the Case II (yield control) scenario, the results showed that in more than 90 percent of the trials, subjects reacted in time to meet the SSD criteria established and, thus, the 2.5-s PRT value was adequate. With respect to Case III scenarios, the PRT was measured from the first head movement after a stop to the application of the accelerator to enter the intersection. The mean and 85th percentile values for all maneuvers combined were 1.82 s and 2.7 s, respectively. The results also showed that the through movement produced a lower value than the mean, while the turning maneuvers produced a higher value. These results to the conclusions that the 2.0-s criteria for Case IIIA be retained and that the PRT value for the Case III turning maneuvers (B and C) be increased from 2.0 to 2.5 s. One other result, which is applicable to the current effort, was that no significant differences were found with respect to age, i.e., increased PRTs were needed to accommodate all drivers.

Fambro, Koppa, Picha, and Fitzpatrick (1998) found significant differences in mean perception-brake response times as a function of age and gender, with older drivers and female drivers demonstrating longer response times. They conducted three separate on-road studies to measure driver perception-brake response time to several SSD situations. Studies were conducted on a closed course, as well as on an open roadway. In one study conducted on the closed course, subjects drove an instrumented test vehicle belonging to the Texas Transportation Institute, and in another closed course study, they drove their own vehicles. In the open roadway study, they drove their own vehicles. Seventeen younger drivers (age 24 or younger) and 21 older drivers (age 55 or older) participated in trials that required them to brake in response to expected and unexpected events, which included a barrel rolling off of a pick-up truck parked next to the roadway, an illuminated LED on the windshield, and a horizontal blockade that deployed ahead of them on the roadway. Across all *expected*-object perception-brake response time trials, the mean response time for younger drivers was 0.52 s and the mean response time for older drivers was 0.66 s. For these "expected" trials, the mean perception-brakeresponse time for males was 0.59 s and for females it was 0.63 s. For the *unexpected*-object perception-brake response trials, longer response times were demonstrated for trials where subjects drove their own vehicles, compared to those in which they drove the Transportation Institute's vehicle. The study authors suggested that subjects were more relaxed and unsuspecting when driving their own vehicles. The mean response time across studies (controlled and open road, own vehicle and research vehicle) for the unexpected object was 1.1 s; the 95th percentile perception-brake response time was 2.0 s.

Based on this finding, Fambro et al. (1998) concluded that AASHTO's 2.5-s perception-brake response time value is appropriate for highway design when SSD is the relevant control. However, they note that at locations or for geometric features where something other than SSD is the relevant control, a different PRT may be appropriate. For example, a longer PRT may be appropriate for intersection or interchange design where more complex decisions and driver speed and/or path correction are required.

Another effort examined the appropriateness of the PRT values currently specified by AASHTO for computing SSD, vehicle clearance interval, sight distance on horizontal curves, and ISD (McGee and Hooper, 1983). With respect to ISD, the results showed the following: for Case I, the driver is not provided with sufficient time or distance to take evasive action if an opposing vehicle is encountered; and for Case II, adequate sight distance to stop before arriving at the intersection is not provided despite the intent of the standard to enable such action. With respect to the PRT values, recommendations include increasing the 2.0-s and 2.5-s values used in Case I and Case II calculations, respectively, to 3.4 s. It was also recommended that the PRT value for Case III scenarios be redefined.

Although there is no consensus from the above studies on the actual values of PRT that should be employed in the ISD computations, there is a very clear concern as to whether the current values are meeting the needs of older drivers. Since older drivers tend to take longer in making a decision, especially in complex situations, the need to further evaluate current PRT values is underscored. Slowed visual scanning of traffic on the intersecting roadway by older drivers has been cited as a cause of near misses of (crossing) crashes at intersections during on-road evaluations. In the practice of coming to a stop, followed by a look to the left, then to the right, and then back to the left again, the older driver's slowed scanning behavior allows approaching vehicles to have closed the gap by the time a crossing maneuver finally is initiated. The traffic situation has changed when the older driver actually begins the maneuver, and drivers on the main roadway are often forced to adjust their speed to avoid a collision. Hauer (1988) stated that "the standards and design procedures for intersection sight triangles should be modified because there is reason to believe that when a passenger car is taken as the design vehicle, the sight distance is too short for many older drivers, who take longer to make decisions, move their heads more slowly, and wish to wait for longer gaps in traffic."

In contrast, recent research conducted by Lerner, Huey, McGee, and Sullivan (1995) concluded that, based on older driver performance, no changes to design PRT values were recommended for ISD, SSD, or decision sight distance (DSD), even though the 85th percentile J values exceeded the AASHTO 2.0-s design standard at 7 of the 14 sites. The J value equals the sum of the PRT time and the time to set the vehicle in motion, in seconds. No change was recommended because the experimental design represented a worst-case scenario for visual search and detection (drivers were required to begin their search only after they had stopped at the intersection and looked inside the vehicle to perform a secondary task). Naylor and Graham (1997), in a field study of older and younger drivers waiting to turn left at stop-controlled intersections (Case IIIB), similarly concluded that the current AASHTO value of 2.0 s is adequate for the PRT (J value) used in calculating ISD at these sites.

Lerner et al. (1995) conducted an on-road experiment to investigate whether the assumed values for Case III driver PRT used in AASHTO design equations adequately represent the range of actual PRT for older drivers. Approximately 33 subjects in each of three driver age groups were studied: ages 20–40, ages 65–69, and age 70 and older. Drivers operated their own vehicles on actual roadways, were not informed that their response times were being measured, and were naive as to the purpose of the study (i.e., they were advised that the purpose of the experiment was to judge road quality and how this relates to aspects of driving). The study included 14 data collection sites on a 90-km- (56-mi-) long route. Results showed that the older drivers *did not* have longer PRT than younger drivers and, in fact, the 85th percentile PRT closely matched the AASHTO design equation value of 2.0 s. The 90th percentile PRT was 2.3 s, with outlying values of 3 to 4 s. The median daytime PRT was approximately 1.3 s. Interestingly, it was found that typical driver actions did not follow the stop/search/decide maneuver sequence implied by the model; in fact, drivers continued to search and appeared ready to terminate or modify their maneuver even after they had begun to move into the intersection. This finding resulted in the study authors' conclusion that the behavioral model on which ISD is based is conservative.

Harwood, Mason, Brydia, Pietrucha, and Gittings (1996) evaluated current AASHTO policy on ISD for Cases I, II, III, IV, and V during performance of NCHRP project 15-14(1), based on a survey of current highway agencies' practices and a consideration of alternative ISD models and computational methodologies, as well as findings from observational studies for selected cases. Although this work culminated in recommendations for minimum distances for the major and minor legs of the sight triangle for all cases, driver age was not included as a study variable; therefore, specific values for these design elements were not included within the recommendations presented in this Handbook, nor is an exhaustive discussion of these materials included in this section. The results of the Harwood et al. (1996) analyses pertaining to ISD for Cases IIIB and IIIC-and by extension for Case V-are of particular interest, however, in the interpretation of other, related findings from an older driver field study in this area. These analysis outcomes are reviewed below.

Prior to the 1990 AASHTO *Green Book*, the issue of ISD for a driver turning left off of a major roadway onto a minor roadway or into an entrance (Case V) was not specifically addressed. In the 1990 *Green Book*, the issue was addressed at the end of the Case III discussions in two paragraphs. In the 1994 *Green Book*, these same paragraphs have been placed under a new condition referred to as Case V. The equation used for determining ISD for Case V was simply taken from the Case IIIA (crossing maneuver at a stop-controlled intersection) and Case IIIB (left-turn maneuver from a stop-controlled minor road onto a major road) conditions, with the primary difference between the cases being the distance traveled during the maneuver. A central issue in defining the ISD for Case V involves a determination of whether the tasks that define ISD for Cases IIIA and IIIB are similar enough to the tasks associated with Case V to justify using the same equation, which follows:

$$ISD = 1.47\ V\ (J + t_a) \qquad \text{English}$$
$$ISD = 0.278\ V\ (J + t_a) \qquad \text{Metric}$$

where:

ISD = intersection sight distance (feet for English equation; meters for metric equation).
V = major roadway operating speed (mi/h for English equation; km/h for metric equation).
J = time required to search for oncoming vehicles, to perceive that there is sufficient time to make the left turn, and to shift gears, if necessary, prior to starting (J is currently assumed to be 2.0 s).
t_a = time required to accelerate and traverse the distance to clear traffic in the approaching lane(s); obtained from figure IX-33 in the AASHTO *Green Book*.

For Case IIIA (crossing maneuver), the sight distance is calculated based on the need to clear traffic on the intersecting roadway on both the left and right sides of the crossing vehicle. For Case IIIB (left turn from a stop), sight distance is based on the requirement to first clear traffic approaching from the left and then enter the traffic stream of vehicles from the right. It has been demonstrated that the perceptual judgments required of drivers in both of these maneuver situations increase in difficulty when opposing through traffic must be considered.

The perceptual task of turning left from a major roadway at an unsignalized intersection or during a permitted signal phase at a signalized intersection requires a driver to make time-distance estimates of a longitudinally moving target as opposed to a laterally moving target. Lateral movement (also referred to as tangential movement) describes a vehicle that is crossing an observer's line of sight, moving against a changing visual background where it passes in front of one fixed reference point after another. Longitudinal movement, or movement in depth, results when the vehicle is either coming toward or going away from the observer. In this case there is no change in visual direction, only subtle changes in the angular size of the visual image, typically viewed against a constant background. Longitudinal movement is a greater problem for drivers because the same displacement of a vehicle has a smaller visual effect than when it moves laterally-that is, lateral movement results in a much higher degree of relative motion (Hills, 1980).

In comparison with younger subjects, a significant decline for older subjects has been reported in angular motion sensitivity. In a study evaluating the simulated change in the separation of taillights indicating the overtaking of a vehicle, Lee (1976) found a threshold elevation greater than 100 percent for drivers ages 70–75, compared with drivers ages 20–29, for brief exposures at night. Older persons may, in fact, require twice the rate of movement to perceive that an object is approaching, versus maintaining a constant separation or receding, given a brief duration (2.0 s) of exposure. In related experiments, Hills (1975) found that older drivers required significantly longer to perceive that a vehicle was moving closer at constant speed: at 31 km/h (19 mi/h), decision times increased 0.5 s between ages 20 and 75. This body of evidence suggests that the 2.0-s PRT (i.e., variable J in the ISD equation above) used for Cases III and V may not be sufficient for the task of judging gaps in opposing through traffic by older drivers. A revision of Case V to determine a minimum required sight-distance value that more accurately reflects the perceptual requirements of the left-turn task may therefore be appropriate.

Harwood et al. (1996) suggested that at locations where left turns from the major road are permitted at intersections and driveways, at unsignalized intersections, and at signalized intersections without a protected turn phase, sight distance along the major road should be provided based on a critical gap approach, as was recommended for left and right turns from the minor road at stop-controlled intersections. The Gap Acceptance model developed and proposed to replace the current ISD AASHTO model is:

$$ISD = 1.47\ VG \quad \text{English}$$
$$ISD = 0.278\ VG \quad \text{Metric}$$

where: ISD = intersection sight distance (feet for English equation; meters for metric equation).
V = operating speed on the major road (mi/h for English equation, km/h for metric equation).
G = the specified critical gap (in seconds); equal to 5.5 s for crossing one opposing lane, plus an additional 0.5 s for each additional opposing lane.

Field data were collected in the NCHRP study to better quantify the gap acceptance behavior of passenger cars and trucks, but only for left- and right-turning maneuvers from minor roadways controlled by a stop sign (Cases IIIB and C). In the Phase I interim report produced during the conduct of the NCHRP project, Harwood et al. (1993) reported that the critical gap currently used by Caltrans is 7.5 s. When current AASHTO Case IIIB ISD criteria are translated to time gaps in the major road traffic stream, the gaps range from 7.5 s (67 m [220 ft]) at a 32-km/h (20-mi/h) operating speed to 15.2 s (475 m [1560 ft]) at a 112-km/h (70-mi/h) operating speed. Harwood et al. (1993) stated that the rationale for gap acceptance as an ISD criterion is that drivers safely accept gaps much shorter than 15.2 s routinely, even on higher speed roadways.

In developing the Gap Acceptance model for Case V, Harwood et al. (1996) relied on data from studies conducted by Kyte (1995) and Micsky (1993). Kyte (1995) recommended a critical gap value of 4.2 s for left turns from the major road by passenger cars for inclusion in the unsignalized intersection analysis procedures presented in the *Highway Capacity Manual* (Transportation Research Board, 1994). A constant value was recommended regardless of the number of lanes to be crossed; however, a heavy-vehicle adjustment of 1.0 s for two-lane highways and 2.0 s for multilane highways were recommended. Harwood et al. (1996) reported that Micsky's 1993 evaluation of gap acceptance behavior for left turns from the major roadway at two Pennsylvania intersections resulted in critical gaps with a 50 percent probability of acceptance (determined from logistic regression) of 4.6 s and 5.3 s. Using the rationale that design policies should be more conservative than operational criteria such as the *Highway Capacity Manual*, Harwood et al. (1996) recommended a critical gap for left turns from the major roadway of 5.5 s, and an increase in the critical gap to 6.5 s for left turns by single-unit trucks and to 7.5 s for left turns by combination trucks. In addition, if the number of opposing lanes to be crossed exceeds one, an additional 0.5 s per additional lane for passenger cars and 0.7 s per additional lane for trucks were recommended.

It is important to note that the NCHRP study did not consider driver age as a variable. However, Lerner et al. (1995) collected judgments about the acceptability of gaps in traffic as a function of driver age for left-turn, right-turn, and through movements at stop-controlled intersections. While noting that these authors found no significant differences between age groups in the *total* time required to perceive, react, and complete a maneuver in a related Case III PRT study, the Lerner et al. (1995) findings indicate that younger drivers accept shorter gaps than older drivers. The 50th percentile gap acceptance point was about 7 s (i.e., if a gap is 7 s long, only about half of the subjects would accept it). The 85th percentile point was approximately 11 s. The oldest group required about 1.1 s longer than the youngest group.

Staplin, Harkey, Lococo, and Tarawneh (1997) conducted an observational field study of driver performance as a function of left-turn lane geometry and driver age at four locations where the main road operating speed was 56 km/h (35 mi/h). The mean left-turn critical gap sizes (in seconds) across all sites, for drivers who had positioned their vehicles within the intersection, were as follows: 5.90 s for young/middle-aged (ages 25–45) females; 5.91 s for the young/middle-aged males; 6.01 s for the young-old (ages 65–74) females, 5.84 s for young-old males; and 6.71 s for the old-old (age 75 and older) females, and 6.55 s for the old-old males. Prominent trends indicated that older drivers demonstrated larger critical gap values at all locations. The young/middle-aged and young-old groups were not significantly different from each other; however, both were significantly different from the old-old group. Critical gap data were not collected in this study for drivers who did not position themselves within the intersection, but it is important to note that the older drivers were less likely to position themselves within the intersection than the young and middle-aged drivers.

Critical gap sizes displayed in a laboratory simulation study in the same project, where oncoming vehicles traveling at 56 km/h (35 mi/h) were viewed on a large screen display in the correct perspective, ranged from 6.4 s to 8.1 s for young/middle-aged drivers and from 5.8 to 10.0 s for drivers age 75 and older. This increase in size and variability of the critical gap for left turns by older drivers suggests that the value for G in the Gap Acceptance model must be increased to accommodate this user group beyond levels recommended in NCHRP 383 (where the performance of older drivers, *per se*, was not at issue).

The culmination of this work was a rigorous exercise of competing models and theoretical approaches for calculating sight distance requirements. As reported by Staplin et al. (1997), current and proposed sight distance models were exercised using data collected in the observational field study. This study was conducted at four intersections that differed in the amount that the opposite left-turn lanes were offset The goal was to determine which model(s), including existing and modified AASHTO models and a Gap Acceptance model, best predicted the data observed in the field.

Several data elements collected in the field received special attention. One of these data elements was the maneuver time of the left-turning driver. This time is equivalent to t_a in the AASHTO model, reference figure IX-33 in the AASHTO (1994) *Green Book*. These times were measured at each of four intersections included in the study for positioned and unpositioned drivers. That is, separate maneuver-time measures were obtained, depending on whether or not the drivers positioned themselves within the intersection prior to turning. Staplin et al. (1997) found no significant differences in maneuver time as a function of age for the drivers turning left at the four intersections studied (which had distances ranging from 26 to 32 m [84 to 106 ft]). Maneuver times for drivers positioned within the intersection versus unpositioned drivers, however, were significantly different. Since older drivers less frequently positioned themselves in the field study, the design value for this factor (maneuver time) should be based on that obtained for unpositioned drivers.

A comparison between AASHTO values and the 95th percentile clearance times demonstrated by positioned drivers and unpositioned drivers in this study is presented in table 6. In table 6, the "positioned" vehicles were located within the intersection, approaching the median or centerline of the cross-street. The "unpositioned" vehicles were at or behind the stop line or end of the left-turn bay. (See figure 8, located in the discussion for Design Element E, for an illustration of driver positioning within an intersection.)

Table 6. Comparison of clearance times obtained in the Staplin et al. (1997) field study with AASHTO *Green Book* values used in sight-distance calculations.

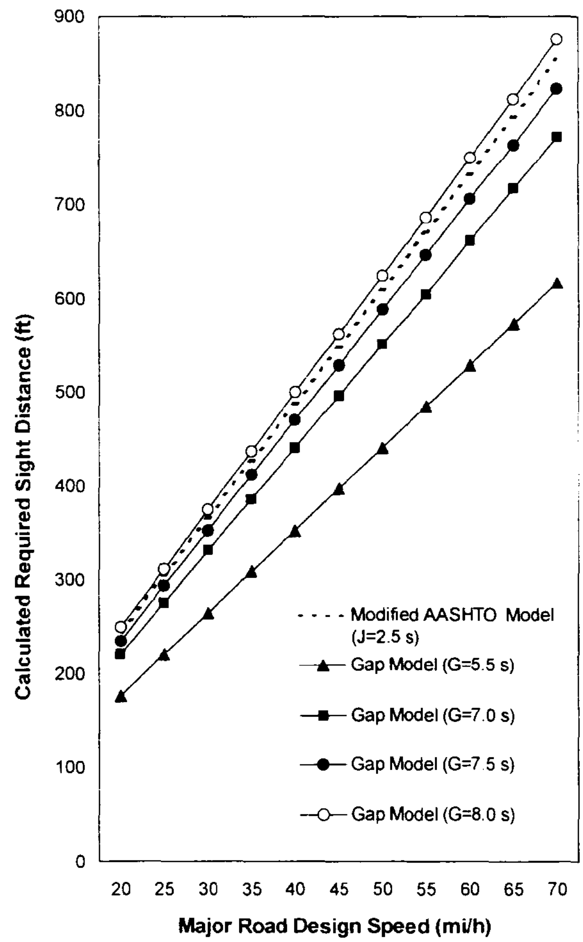
Measure	**Vehicle Location**	**Left-Turn Lane Geometry**			
		-4.3 m (-14 ft) Offset	**-1 m (-3 ft) Offset**	**0 m (0 ft) Offset**	**+1.8 m (+6 ft) Offset**
Distance Traveled, m(ft)	Positioned	21.3 m (70 ft)	20.4 m (67 ft)	19.5 m (64 ft)	21.3 m (70 ft)
95th Percentile Clearance Time (s) From Field Study	Positioned	3.8 s	3.9 s	3.9 s	3.9 s
AASHTO Clearance Time (s) From Figure IX-33	Positioned	5.1 s	5.0 s	5.0 s	5.1 s
Distance Traveled, m(ft)	Unpositioned	32.3 m (106 ft)	29.9 m (98 ft)	25.6 m (84 ft)	26.8 m (88 ft)
95th Percentile Clearance Time (s) From Field Study	Unpositioned	6.7 s	6.4 s	6.6 s	5.7 s
AASHTO Clearance Time (s) From Figure IX-33	Unpositioned	6.5 s	6.2 s	5.9 s	6.0 s

A detailed discussion of the outputs from the model exercise is provided in the publication *Intersection Geometric Design and Operational Guidelines for Older Drivers and Pedestrians* (Staplin, Harkey, Lococo, and Tarawneh, 1997). However, the most significant result for the purposes of this discussion is as follows: *the required sight distances computed using a modified AASHTO model (where the PRT was increased to 2.5 s) produced values that were most predictive of actual field operations.*

Thus, when ISD is calculated using the AASHTO model as it relates to drivers turning left from a major roadway, there is evidence that the PRT value should be increased to 2.5 s to provide adequate sight distance. The Gap Acceptance model, on the other hand, produced sight distance values that were approximately 23 percent shorter than the current AASHTO model, which uses a PRT of only 2.0 s. If the Gap Acceptance model is going to be used, particularly where there are significant volumes of older left-turning drivers, an adjustment factor applied to increase the sight distance to better accommodate this driver age group appears warranted.

To determine what adjustment is most appropriate in this regard, a set of analyses were performed in which the goal was to identify a value of G that would yield required sight distance values meeting or exceeding those derived from the modified AASHTO model formula (i.e., where J = 2.5 s). By extension, this result would also best match the behavior of drivers 75 and older observed in the field study.

Very simply, alternate values for G were substituted into the gap formula for calculating minimum required sight distance (1.47VG). These included 5.5 s, as recommended by Harwood et al. (1996), as well as values which increase in 0.5-s increments. The results of these calculations for alternate values of G, beginning at 7.0 s, are plotted against the required sight distance calculated using the modified AASHTO formula [$1.47V(J+t_a)$; where J=2.5 s and t_a is obtained from table IX-33 in the *Green Book*] in figure 5. As shown in this figure, a gap of 8.0 s affords a sight distance for left-turning drivers that equals or exceeds the requirements calculated using the modified AASHTO model for major road design speeds from 32 km/h to 113 km/h (20 mi/h to 70 mi/h).



1 ft =0.305 m
1 mi/h = 1.61 km/h

Figure 5. Comparison of required intersection sight-distance values calculated using modified AASHTO model (J=2.5 s) and Gap Acceptance model using varying values for G.

E. Design Element: Offset (Single) Left-Turn Lane Geometry, Signing, and Delineation

Table 7. Cross-references of related entries for offset (single) left-turn lane geometry, signing, and delineation.

Applications in Standard Reference Manuals			
MUTCD (2000)	AASHTO *Green Book* (1994)	NCHRP 279, *Intersection Channelization Design Guide* (1985)	*Traffic Engineering Handbook* (1999)
Sect. 1A.13, *median, regulatory signs, road delineators, stop line, & wrong-way arrows* Sect. 1A.14, *Abbreviations* Table 2B-1 Sects. 2A.24, 2B.3, 2B.29, 2B.30, 2B.32, 2B.33 & 2E.50 Figs. 2A-2 through 2A-6, 2E-31 and 2E-32 Sect. 3B.4 Fig. 3B-11 a, b & d Fig. 3B-21 Sect. 3B.11 Sects. 3B.16 & 3B.19 Figs. 3B-19, 3B-21 & 3B-22 Sects. 3B.21, 3C.03, 3D.03, 3E.01, 3G.04 through 3G.06	p. 45, Para. 1 pp. 679-687, Sects. on *Island Size and Designation & Delineation and Approach-End Treatment* pp. 783-787, Sects. on *Median Left-Turn Lanes & Median End Treatment*	p. 1, 1st bullet p. 3, 2nd col., Para. 5 p. 6, Table 2-1 p. 10, Table 2-4 & 2nd col., Para. 3 p. 14, Sect. on *Decision Sight Distance* p. 17, Middle fig. p. 29, Para. 1 p. 34, Para. 1 and top fig. p. 35, Paras. 2-3 p. 60, Middle fig.	pp. 375-376, Sect. on *Intersection Sight Distance (ISD)* p. 386, Para. 4 p. 388, Para. 2

Studies examining older driver crashes and the types of maneuvers being performed just prior to the collision have consistently found this group to be overinvolved in left-turning crashes at both rural and urban signalized intersections and have indicated that failure to yield the right-of-way (as the turning driver) was the principal violation type (Staplin and Lyles, 1991; Council and Zegeer, 1992). Underlying problems for the maneuver errors include the misjudgment of oncoming vehicle speed, misjudgment of available gap, assuming the oncoming vehicle was going to stop or turn, and simply not seeing the other vehicle. Joshua and Saka (1992) noted that sight distance problems at intersections, which result from queued vehicles in opposite left-turn lanes, pose safety and capacity deficiencies, particularly for unprotected (permitted) left-turn movements. These researchers found a strong correlation between the offset for opposite left-turn lanes—i.e., the distance from the inner edge of a left-turn lane to the outer edge of the opposite left-turn lane—and the available sight distance for left-turning traffic.

The alignment of opposite left-turn lanes and the horizontal and vertical curvature on the approaches are the principal geometric design elements that determine how much sight distance is available to a left-turning driver. Operationally, vehicles in the opposite left-turn lane waiting to turn left can also restrict the (left-turning) driver's view of oncoming traffic in the through lanes. The level of blockage depends on how the opposite left-turn lanes are aligned with respect to each other, as well as the type/size of vehicles in the opposing queue. Restricted sight distance can be minimized or eliminated by offsetting opposite left-turn lanes so that left-turning drivers do not block each other's view of oncoming through traffic. When the two left-turn lanes are exactly aligned, the offset distance has a value of zero. Negative offset describes the situation where the opposite left-turn lane is shifted to the left. Positive offset describes the situation where the opposite left-turn lane is shifted to the right. Figure 6 illustrates the relationships between the opposite left-turn lanes for negative and positive offset lane geometry. Positive offset left-turn lanes and aligned left-turn lanes provide greater sight distances than negative offset left-turn lanes, and a positive offset provides greater sight distance than the aligned configuration.

Older drivers may experience greater difficulties at intersections as the result of diminished

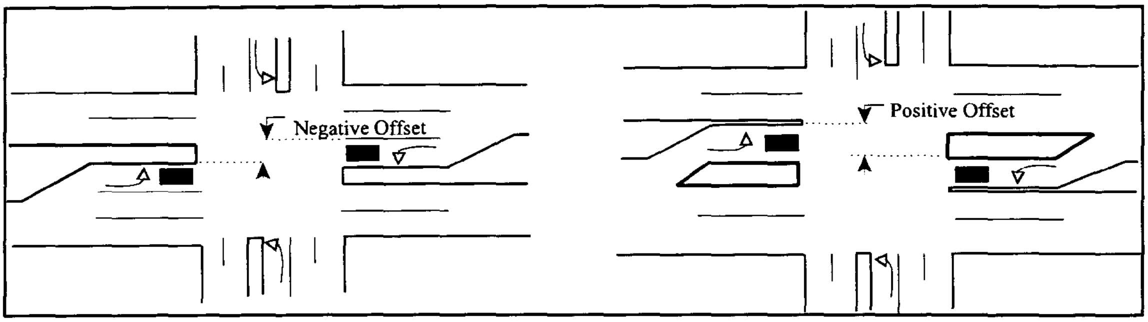


Figure 6. Relationship of left-turn lanes for negative and positive offset geometry.

visual capabilities such as depth and motion perception, as well as diminished attention-sharing (cognitive) capabilities. Studies have shown that there are age differences in depth and motion perception. Staplin, Lococo, and Sim (1993) found that the angle of stereopsis (seconds of arc) required for a group of drivers age 75 and older to discriminate depth using a commercial vision tester was roughly twice as large as that needed for a group of drivers ages 18 to 55 to achieve the same level of performance. However, while accurate perception of the distance to geometric features delineated at intersections—as well as to potentially hazardous objects such as islands and other raised features—is important for the safe use of these facilities, relatively greater attention by researchers has been placed upon motion perception, where dynamic stimuli (usually other vehicles) are the primary targets of interest. It has been shown that older persons require up to twice the rate of movement to perceive that an object is approaching, and they require significantly longer to perceive that a vehicle is moving closer at a constant speed (Hills, 1975). A study investigating causes of older driver overinvolvement in turning crashes at intersections, building on the previously reported decline for detection of angular expansion cues, did not find evidence of overestimation of time-to-collision (Staplin et al., 1993). At the same time, a relative insensitivity to approaching (conflict) vehicle speed was shown for older versus younger drivers; this result was interpreted as supporting the notion that older drivers rely primarily or exclusively on perceived *distance*—not time or velocity—to perform gap acceptance judgments, reflecting a reduced ability to integrate time and distance information with increasing age. Thus, a principal source of risk at intersections is the error of an older, turning driver when judging gaps in front of fast vehicles.

Several recent studies examining the minimum required sight distance for a driver turning left from a major roadway to a minor roadway, as a function of major road design speed, have provided data necessary to determine: (1) the left-turn lane offset value needed to achieve the minimum required sight distance, and (2) the offset value that will provide unlimited sight distance. A fundamental premise in these studies, which are described below, is that it is not the amount of left-turn lane offset *per se*, but rather the sight distance that a given level of offset provides that should be the focus of any recommendations pertaining to the design of opposite left-turn lanes.

In a study conducted by McCoy, Navarro, and Witt (1992), guidelines were developed for offsetting opposite left-turn lanes to eliminate the left-turn sight distance problem. All minimum offsets specified in the guidelines are positive. For 90-degree intersections on level tangent sections of four-lane divided roadways with 3.6-m- (12-ft-) wide left-turn lanes in 4.9-m- (16-ft-) wide medians with 1.2-m- (4-ft-) wide medial separators, the following conclusions were stated by McCoy et al. (1992): (1) a 0.6-m (2-ft) positive offset provides unrestricted sight distance when the opposite left-turn vehicle is a passenger car, and (2) a 1.06-m (3.5-ft) positive offset provides unrestricted sight distance when the opposite left-turn vehicle is a truck, for design speeds up to 113 km/h (70 mi/h).

Harwood, Pietrucha, Wooldridge, Brydia, and Fitzpatrick (1995) conducted an observational field study and a crash analysis to develop design policy recommendations for the selection of median width at rural and suburban divided highway intersections based on operational and safety considerations. They found that at rural unsignalized intersections, both crashes and undesirable driving behaviors decrease as median width increases. However, at suburban signalized and unsignalized intersections, crashes and undesirable behaviors increase as the median width increases. At suburban intersections, it is therefore suggested that the median generally should not be wider than necessary to accommodate pedestrians and the appropriate median left-turn treatment needed to serve current and anticipated future traffic volumes. Harwood et al. stated that wider medians generally have positive effects on traffic operations and safety; however, wider medians can result in sight restrictions for left-turning vehicles due to the presence of opposite left-turn vehicles. The most common solution to this problem is to offset the left-turn lanes, using either parallel offset or tapered offset left-turn lanes.

Figure 7 compares conventional left-turn lanes with these two alternative designs. As noted by Harwood et al. (1995), parallel and tapered offset left-turn lanes are still not common, but are used increasingly to reduce the risk of crashes due to sight restrictions from opposite left-turn vehicles. Parallel offset left-turn lanes with 3.6-m (12-ft) widths can be constructed in raised medians with widths as narrow as 7.2 m (24 ft), and can be provided in narrower medians if restricted lane widths or curb offsets are used or a flush median is provided (Bonneson, McCoy, and Truby, 1993). Tapered offset left-turn lanes generally require raised medians of 7.2 m (24 ft) or more in width.

For separation of the left-turn lane from through traffic in alternative designs such as those discussed above, the practitioner must choose between raised channelization and channelization accomplished through the use of pavement markings. As noted earlier, left-turn channelization separating through and turning lanes may, because of its placement, constitute a hazard when a raised treatment is applied, especially on high-speed facilities. Detection and avoidance of such hazards requires visual and response capabilities known to decline significantly with advancing age, supporting recommendations for treatments to improve the contrast for these channelizing features at intersections (see Design Element C).

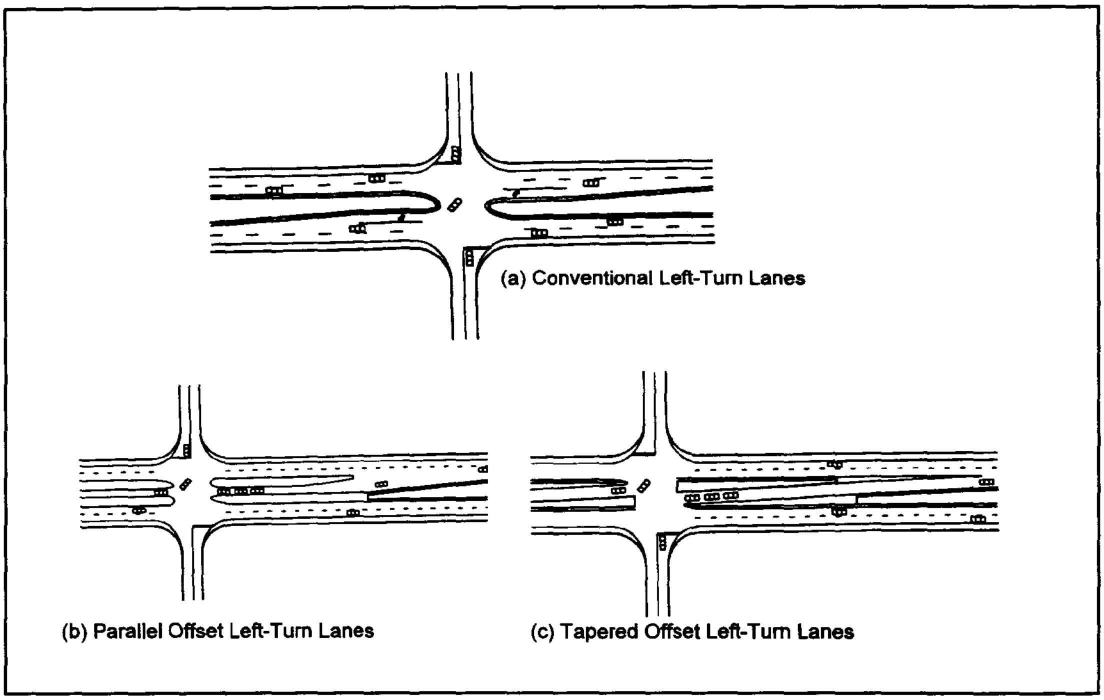


Figure 7. Alternative left-turn treatments for rural and suburban divided highways.
Source: Bonneson, McCoy, and Truby (1993).

As discussed in some detail under Design Element D, Staplin, Harkey, Lococo, and Tarawneh (1997) performed a laboratory study, field study, and sight distance analysis to measure driver age differences in performance under varying traffic and operating conditions as a function of varying degrees of offset of opposite left-turn lanes at suburban arterial intersections. Research findings indicated that an increase in sight distance through positively offsetting left-turn lanes can be beneficial to left-turning drivers, particularly older drivers. In the field study, where left-turn vehicles needed to cross the paths of two or three lanes of conflicting traffic (excluding parking lanes) at 90-degree, four-legged intersections, four levels of offset of opposite left-turn lane geometry were examined. These levels include: (1) 1.8-m (6-ft) "partial positive" offset, (2) aligned (no offset) left-turn lanes, (3) 0.91-m (3-ft) "partial negative" offset, and (4) 4.3-m (14-ft) "full negative" offset. All intersections were located within a growing urban area where the posted speed limit was 56 km/h (35 mi/h). In addition, all intersections were controlled by traffic-responsive semi-actuated signals, and all left-turn maneuvers were completed during the permitted left-turn phase at all study sites.

In the analysis of the field study lateral positioning data, it was found that the partial positive offset and aligned locations had the same effect on the lateral positioning behavior of drivers. Drivers moved approximately 1.5 m (5 ft) to the left when there was a large negative offset, clearly indicating that sight distance was limited. There was a significant difference between the partial negative offset geometry and the partial positive offset or aligned geometries, suggesting a need for longer sight distance when opposite left-turn lanes are even partially negatively offset. The fact that older drivers (and females) were less likely to position themselves (i.e., pull into the intersection) in the field studies highlights the importance of providing adequate sight distance for unpositioned drivers for all left-turn designs. Vehicle positioning refers to the location within an intersection at which a left-turning vehicle waits for an acceptable gap in the opposing through-traffic stream; specifically at issue is the positioning behavior of drivers attempting to make a left turn through the conflicting through traffic while being opposed or blocked by at least one vehicle trying to make a left-turn maneuver from the opposite direction. The restriction of sight distance for an unpositioned versus a positioned driver at an intersection with aligned left-turn lanes is shown in figure 8.

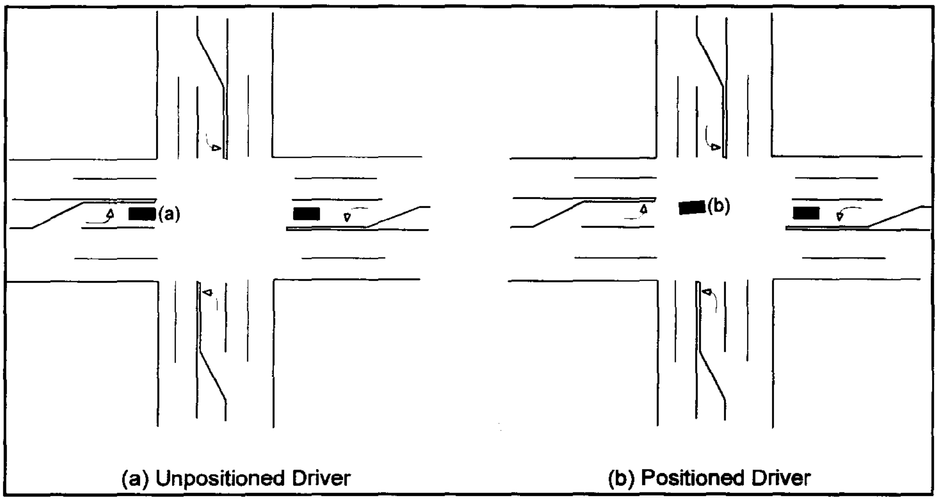
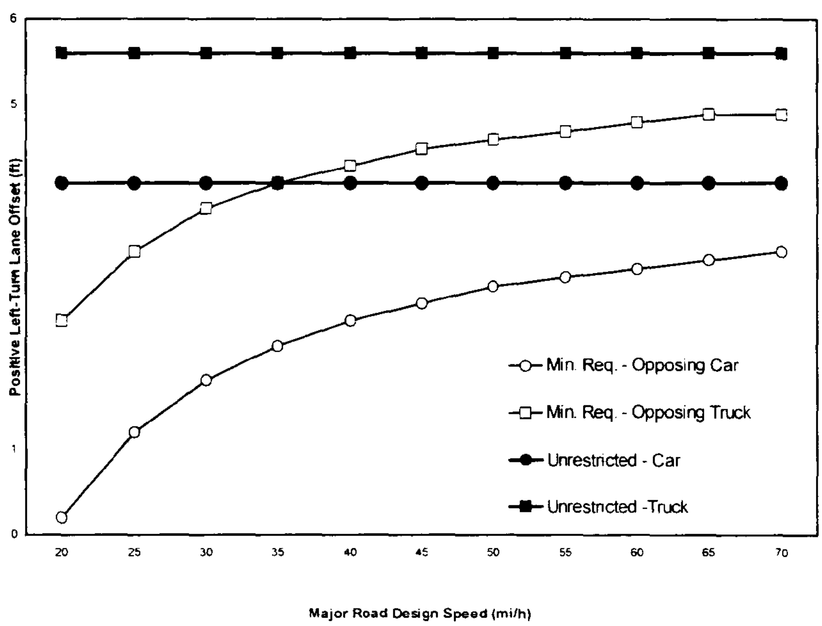


Figure 8. Difference in sight-distance restriction for an unpositioned driver and a positioned driver at an aligned intersection with an opposing left-turning driver.

Several issues were raised in the research conducted by Staplin et al. (1997) regarding the adequacy of the current and proposed ISD models for a driver turning left from a major roadway. The researchers exercised alternative sight distance models, including the current AASHTO Case V model using 2.0 s for the PRT, a modified AASHTO model using a 2.5-s PRT, and a Gap Acceptance model proposed in NCHRP 383 by Harwood, Mason, Brydia, Pietrucha, and Gittings (1996). The proposed Gap Acceptance model relies on a critical gap value in place of PRT and maneuver time. A detailed description of the model parameters and output can be found in the FHWA report, *Intersection Geometric Design and Operational Guidelines for Older Drivers and Pedestrians* (Staplin et al., 1997). Of particular significance was the finding that the modified AASHTO model with the longer PRT of 2.5 s was the model most predictive of actual field operations. Also of significance was the dramatic decrease in required sight distance that occurred with the Gap Acceptance model compared to the traditional AASHTO model. Across all intersections and all design speeds, the required sight distance was approximately 23 percent less using the Gap Acceptance model. However, this was expected since the rationale behind the use of a Gap Acceptance model (see Harwood et al., 1996) in place of the current AASHTO model is the fact that drivers are commonly observed accepting shorter gaps than those implied by the current model. As discussed under Design Element D, subsequent analyses established a recommendation for use of an 8.0-s gap size (plus 0.5 s for each additional lane crossed) to adjust the Gap Acceptance model to accommodate older driver needs for increased sight distance.

Regardless of which model is used to compute ISD for drivers turning left off a major roadway, a practical countermeasure to increase the sight distance is through positive offset of left-turn lanes. As shown in the study by Staplin et al. (1997), such designs result in significantly better performance on the part of all drivers, but especially for older drivers. Prior work by McCoy et al. (1992) examined the issue of offset left-turn lanes and developed an approach that could be used to compute the amount of offset that is required to minimize or eliminate the sight restriction caused by opposing left-turn vehicles.

This approach, incorporating the parameters represented in the intersection diagram shown earlier in figure 4 (see Design Element D), was applied to the intersections in the study by Staplin et al. (1997) to determine the amount of offset that would be required when using the modified AASHTO model (i.e., J = 2.5 s). The left-turn lane offsets required to achieve the minimum required sight distances calculated using this model are shown in figure 9, in addition to the offsets required to provide unrestricted sight distance. Based on intersections examined in the study, the offset necessary to achieve unrestricted sight distance for opposing left-turning cars is 1.2 m (4.1 ft) and for opposing left-turning trucks is 1.7 m (5.6 ft).



1 ft = 0.305 m
1 mi/h = 1.61 km/h

Figure 9. Left-turn lane offset design values necessary to achieve unrestricted sight distances and minimum required sight distances calculated using either the modified AASHTO model (J=2.5 s) *or* the Gap Acceptance model with G=8.0 s.

Finally, the potential for wrong-way maneuvers, particularly by older drivers, at intersections with positive offset channelized left-turn lanes was raised during a panel meeting comprised of older driver experts and highway design engineers during conduct of the research performed by Staplin et al. (1997). The concern expressed was that drivers turning left from the minor road may turn too soon and enter the channelized left-turn lane, instead of turning around both medians. Similar concern was raised by highway engineers surveyed by Harwood et al. (1995) during conduct of NCHRP project 15-14(2). These authors reported that the potential for wrong-way movements by opposing-direction vehicles entering the left-turn roadway is minimal if proper signing and pavement markings are used.

Researchers studying wrong-way movements at intersections—particularly the intersection of freeway exits with secondary roads—have found that such movements resulted from left-turning vehicles making an early left turn rather than turning around the nose of the median, and have proposed and tested several countermeasures. Scifres and Loutzenheiser (1975) reported that indistinct medians are design elements that reduce a driver's ability to see and understand the overall physical and operational features of an intersection, increasing the frequency of wrong-way movements. They suggested delineation of the median noses to increase their visibility and improve driver understanding of the intersection design and function. Also, increasing the conspicuity of ONE WAY, WRONG WAY, and DO NOT ENTER signs by using larger-than-standard (*MUTCD*) size signs, and using retroreflective sheeting on these signs that provides for high brightness at the wide observation angles typical of the sign placements and distances at which these signs are viewed (e.g., 1.0+ degrees) will be of benefit to drivers, particularly those with age-related diminished visual and attentional capabilities. Parsonson and Marks (1979) found that the use of the two-piece, 7.1-m- (23.5-ft-) long arrow pavement marking (wrong-way arrow) was effective in preventing wrong-way entries onto freeway exit ramps in Georgia. Later work in this State found a benefit of pulling the nose back from the intersection and extending the median line from the nose to the intersection using painted markings and raised retroreflectors; this treatment reduced the frequency of impacts with the median by turning vehicles, particularly trucks.[1]

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[1] As per feedback provided by State engineers during a training workshop conducted by Handbook authors on August 6-7, 1998.

F. Design Element: Treatments/Delineation of Edgelines, Curbs, Medians, and Obstacles

Table 8. Cross-references of related entries for treatments/delineation of edgelines, curbs, medians, and obstacles.

Applications in Standard Reference Manuals				
***MUTCD* (2000)**	**AASHTO *Green Book* (1994)**	***Roadway Lighting Handbook* (1978)**	**NCHRP 279, *Intersection Channelization Design Guide* (1985)**	***Traffic Engineering Handbook* (1999)**
Sect. 1A.13, *edgeline markings, island, & object markers* Sect. 3A.06 Sects. 3B.09, 3B.10, 3B.11, 3B.13, & 3B.21 Sects. 3C.01 through 3C.03 Sect. 3E.01 Sect. 3F.02 Sects. 3G.01 through 3G.06	p. 45, Para. 1 p. 314, Para. 7 p. 315, Para. 1 pp. 344-348, Sect. on *Types of Curbs* p. 347, Para. 5 p. 348, Paras. 1-3 p. 475, Para. 6 p. 519, Para. 2 p. 637, Para. 7 p. 639, Fig. IX-7a pp. 679-689, Sects. on *Island Size and Designation, Delineation and Approach End Treatment, & Right-Angle Turns With Corner Islands* p. 755, Sect. on *Shape of Median End* pp. 756 & 761-763, Figs. IX-59 through IX-62 p. 783, Paras. 2-4 pp. 785-786, Figs. IX-73 & IX-74 pp. 786-787, Sect. on *Median End Treatment*	p. 2, 2nd col. Para. 1 p. 3, Para. 4 p. 4, 1st bullet p. 9, Sect. on *Contrast* p. 17, Form 1 p. 21, Table 1 p. 24, Example Form 1 pp. 29-30, Sect. on *Adverse Geometry and Environment Warrant* *p. 31*, Item *A*	p. 24, Para. 1 p. 35, Para. 2 & bottom left fig. p. 39, All figs. p. 66, 2nd col., Para. 1 pp. 69 & 75, Sects. on *Traffic Islands & Guidelines for Selection of Island Type* p. 74, Fig. 4-31 p. 76, Item 1 pp. 102-103, Intersect. No. 8	p. 434, Sect. on *Edge (Fog) Lines* p. 436, Para. 2 p. 438, Item 5 p. 439, Sect. on *Obstruction Approach* p. 440, Paras. 5 & 7

The discrimination at a distance of gross highway features, as opposed to the fine detail contained in a sign message, governs drivers' perceptions of intersection geometric elements. Thus, the conspicuity of such elements as curbs, medians, and obstacles, as well as all raised channelization, is of paramount importance in the task of safely approaching and choosing the correct lane for negotiating an intersection, as well as avoiding collisions with the raised surfaces. During the conduct of their driving task analysis, McKnight and Adams (1970a, 1970b) identified five driving tasks related specifically to the conspicuity of intersection geometric elements: (1) maintain correct lateral lane position, (2) survey pavement markings, (3) survey physical boundaries, (4) determine proper lane position for the intended downstream maneuver, and (5) search for path guidance cues. The visual/perceptual requirement common to the performance of these tasks is contrast sensitivity for detecting lane lines, pavement word and symbol markings, curbs and roadway edge features, and median barriers.

Older drivers' decreased contrast sensitivity, reduced useful field of view, increased decision time—particularly in response to unexpected events—and slower vehicle control movement execution combine to put these highway users at greater crash risk when approaching and negotiating intersections. The smaller the attentional demand required of a driver to maintain the correct lane position for an intended maneuver, the greater the attentional resources available for activities such as the recognition and processing of traffic control device messages and detection of conflict vehicles and pedestrians.

A variety of conspicuity-enhancing treatments are mandated in current practice. The *MUTCD* (section 3B.10, Approach Markings for Obstructions) specifies that pavement markings shall be used to guide traffic away from fixed objects (such as median islands and channelization islands) within a paved roadway. Section 3B.21 (Curb Markings) states that retroreflective solid yellow markings should be placed on the curbs of islands that are located in the line of traffic flow where the curb serves to channel traffic to the right of the obstruction, and that retroreflective solid white markings should be used (on curbs) when traffic may pass on either side of the island. Section 3E.01 (Colored Pavements) describes the use of colored pavements as traffic control devices, where yellow shall be used for median islands and white for channelizing islands, and section 3G.03 (Island Marking Application) describes the use of pavement and curb markings, object markers, and delineators for island marking application. Supplementary treatments and requirements for in-service brightness levels for certain elements contained in existing guidelines are presently an issue.

The conspicuity of curbs and medians besides aiding in the visual determination of how an intersection is laid out is especially important when medians are used as pedestrian refuges. Care must be taken to ensure that pedestrian refuges are clearly signed and made as visible as possible to passing motorists.

Research findings describing driver performance differences directly affecting the use of pavement markings and delineation focus upon (age-related) deficits in spatial vision. In a pertinent laboratory study conducted by Staplin, Lococo, and Sim (1990), two groups of subjects (ages 19–49 and 65–80) viewing a series of ascending and descending brightness delineation targets were asked to report when they could just detect the direction of roadway curvature at the horizon (roadway heading)—left versus right—from simulated distances of 30.5 m (100 ft) and 61 m (200 ft). Results showed that the older driver group required a contrast of 20 percent higher than the younger driver group to achieve the discrimination task in this study.

To describe the magnitude of the effects of age and visual ability on delineation detection/recognition distance and retroreflective requirements for threshold detection of pavement markings, a series of analyses using the Ford Motor Company PC DETECT computer model (see Matle and Bhise, 1984) yielded the stripe contrast requirements shown in table 9. PC DETECT is a headlamp sight distance model that uses the Blackwell and Blackwell (1971, 1980) contrast sensitivity formulations to calculate the distance at which various types of targets illuminated by headlamps first become visible to approaching drivers, with and without glare from opposing headlights. The top 5 percent of 25-year-olds (the best-performing younger drivers) and bottom 5 percent of 75-year-olds (the worst-performing older drivers) were compared in this analysis.

Blackwell and Taylor (1969) conducted a study of surface pavement markings employing an interactive driving simulator, plus field evaluations. They concluded that driver performance —measured by the probability of exceeding lane limits—was optimized when the perceived brightness contrast between pavement markings and the roadway was 2.0. A study by Allen, O'Hanlon, and McRuer (1977) also concluded that delineation contrast should be maintained above a value of 2.0 for adequate steering performance under clear nighttime driving conditions. In other words, because contrast is defined as the difference between target and background luminance, divided by the background luminance alone, these studies have asserted that markings must appear to be *at least* three times as bright as the road surface. Also, because these studies were not specifically focused on the accommodation of older drivers—particularly the least capable members of this group—the contrast requirements defined in more recent modeling studies and analyses, as presented in table 9, are accorded greater emphasis. Taking the indicated value for the least capable 5 percent of 75-year-olds into account, as well as the prior field evaluations, a contrast requirement of 3.0 for pavement markings appears most reasonable.

Important note: Whether luminance is measured in metric or English units [candelas per square meter (cd/m²) or footlamberts (fL)], contrast is a dimensionless number; thus, the present recommendations, as well as the calculation of contrast level, are independent of the unit of measurement.

Table 9. Contrast requirements for edgeline visibility at 122 m (400 ft) with a 5-s preview at a speed of 88 km/h (55 mi/h), as determined by PC DETECT computer model.

Driver Age Group/ % Accommodated	Worst-Case Glare	No Glare
Age 25 / top 5%	0.11	0.05
Age 75 / bottom 5%	7.21	3.74

Finally, inadequate conspicuity of raised geometric features at intersections has been brought to the attention of researchers during the conduct of several focus group studies involving older drivers. Subjects reported difficulty knowing where to drive, due to missing or faded roadway lines on roadway edges and delineation of islands and turning lanes. They also reported hesitating during turns because they did not know where to aim the vehicle (Staplin, Lococo, and Sim, 1990). In another focus group, subjects suggested that the placement of advance warning pavement markings be located as far in advance of an intersection as practicable (Council and Zegeer, 1992). Drivers ages 66–77 and older who were participating in focus group discussions conducted by Benekohal, Resende, Shim, Michaels, and Weeks (1992) reported that intersections with too many islands are confusing because it is hard to find which island the driver is supposed to go around. Raised curbs that are unmarked are difficult to see, especially in terms of height and direction, and this results in people running over them. These older drivers stated that they would prefer to have rumble strips in the pavement to warn them of upcoming concrete medians and to warn them about getting too close to the shoulder. In more recent focus group discussions conducted to identify intersection geometric design features that pose a difficulty for older drivers and pedestrians (Staplin, Harkey, Lococo, and Tarawneh, 1997), drivers mentioned that they have problems seeing concrete barriers in the rain and at night, and characterized barriers as "an obstruction waiting to be hit."

An inventory of the materials and devices commonly employed to delineate roadway edges, curbs, medians, and obstacles includes: retroreflective paint or tape, raised pavement markers (RPMs), post-mounted delineators (PMDs), object markers, and chevron signs.

G. Design Element: Curb Radius

Table 10. Cross-references of related entries for curb radius.

Applications in Standard Reference Manuals			
AASHTO *Green Book* (1994)	NCHRP 279, *Intersection Channelization Design Guide* (1985)		*Traffic Engineering Handbook* (1999)
pp. 665-672, Sect. on *Effect of Curb Radii on Turning Paths* p. 675, Para. 2 pp. 752-755, Sect. on *Control Radius for Minimum Turning Paths* pp. 758-763, Sect. on *Median Openings Based on Control Radii for Design Vehicles Noted*	p. 1, 2nd bullet p. 6, Paras. 4-5 p. 10, Table 2-4 p. 20, Bottom fig. p. 21, 2nd col, Item 4 & Fig. 3-1 p. 22, 2nd fig from bottom of pg. p. 23, Bottom right fig. p. 26, Bottom fig. p. 35, Para. 2 p. 36, Middle fig. & associated notes p. 38, Middle fig. & associated notes pp. 66-69, Sects. on *Corner Radius Design & Radius of Turn*	pp. 70-73, Figs. 4-27 through 4-29 p. 77, Fig. 4-32 p. 83, Sects. on *Driveways Along Major Arterials and Collectors & Consideration of Pedestrians* pp. 84-87, Figs. 4-37 through 4-39 pp. 93-94, Intersect. No. 2 pp. 96-97, Intersect. No. 4 pp. 122-125, Intersect. Nos. 18-19 pp. 128-129, Intersect. No. 20B pp. 132-135, Intersect. Nos. 22-23	p. 355, Paras. 5-6 p. 356, Table 11-5 p. 358, Table 11-6 p. 387, Sect. on *Corner Radius Design* p. 409, Para. 3

Recommendations for this design element address the radius of the curve that joins the curbs of adjacent approaches to an intersection. The size of the curve radius affects the size of the vehicle that can turn at the intersection, the speed at which vehicles can turn, and the width of the intersection that must be crossed by pedestrians. If curb radii are too small, lane encroachments resulting in traffic conflicts and increased crash potential can occur. If the radii are too large, pedestrian exposure may be increased (although, if large enough, refuge islands may be provided). The procedures used in the design of curb radii are well detailed in the *Green Book* (AASHTO, 1994).

McKnight and Stewart (1990) identified the task of positioning a vehicle in preparation for turning as a critical competency. A significant problem identified in a task analysis to prioritize older drivers' problems with intersections is carrying out the tight, right-turn maneuver at normal travel speed on a green light (Staplin, Lococo, McKnight, McKnight, and Odenheimer, 1994). The problems are somewhat moderated when right turns are initiated from a stop, because the turn can be made more slowly, which reduces difficulties with short radii. Older drivers may seek to increase the turning radius by moving to the left before initiating the turn, often miscommunicating an intent to turn left and encouraging following drivers to pass on the right. Or, they may initiate the turn from the correct position, but swing wide into a far lane in completing the turn in order to lengthen the turning radius and thus minimize rotation of the steering wheel. Encroaching upon a far lane can lead to conflict with vehicles approaching from the right or, on multilane roads, oncoming drivers turning to their left at the same time. The third possibility is to cut across the apex of the turn, possibly dragging the rear wheels over the curb. Each of these shortcomings in lanekeeping can be overcome by a channelized right-turn lane or wider curb radii.

Chu (1994) found that relative to middle-aged drivers (ages 25–64), older drivers (age 65 and older) tend to drive larger automobiles and drive at slower speeds. Although large heavy cars are associated with a crash fatality rate that is less than one-quarter of that associated with the smallest passenger cars (Insurance Institute for Highway Safety, 1991) and are, therefore, a wise choice for older drivers who are more frail than their middle-aged counterparts, large vehicles have larger turning radii, which may exacerbate the problems older drivers exhibit in lanekeeping during a turn. Roberts and Roberts (1993) reported that common arthritic illnesses such as osteoarthritis, which affects more than 50 percent of the elderly population, and rheumatoid arthritis, affecting 1 to 2 percent, are relevant to the tasks of turning and gripping the steering wheel. A hand deformity caused by either osteoarthritis or rheumatoid arthritis may be very sensitive to pressure, making the driver unwilling to apply full strength to the steering wheel or other controls. In an assessment of 83 drivers with arthritis, Cornwell (1987) found that 83 percent of the arthritic group used both hands to steer, 7 percent used the right hand only, and 10 percent the left hand only; in this study, more than one-half of the arthritic group required steering modifications, either in the form of power steering or other assistive devices, such as a smaller steering wheel.

The *Intersection Channelization Design Guide* (Neuman, 1985) states that intersections on high-speed roadways with smooth alignment should be designed with sufficient radii to accommodate moderate- to high-speed turns. At other intersections, such as in residential neighborhoods, low-speed turns are desirable and smaller corner radii are appropriate in these cases. In addition, selection of a design vehicle is generally based on the largest standard or typical vehicle type that would regularly use the intersection. For example, a corner radius of 15 m (50 ft) will accommodate moderate-speed turns for all vehicles up to WB-50 (combination truck/large semitrailer with an overall length of 17 m [55 ft]). However, many agencies are designing intersections along their primary systems to accommodate a 21-m (70-ft), single-trailer design vehicle (C-70). Table 4-8 (p. 66) of the *Intersection Channelization Design Guide* provides guidelines for the selection of a design vehicle. It further specifies in table 4-9 what the operational characteristics are of various corner radii. For example, a corner radius of less than 1.5 m (5 ft) is not appropriate, even for P design vehicles (passenger cars), whereas a corner radius of 6 to 9 m (20 to 30 ft) will accommodate a low-speed turn for P vehicles and a crawl-speed turn for sport utility (SU) vehicles (single-unit truck, 9 m [30 ft] in length) with minor lane encroachment.

Of equal importance to the consideration of the right-turning design vehicle in determining curb radii is the consideration of pedestrian crossing time, particularly in urban areas. Smaller corner radii (less than 9 m [30 ft]) can decrease right-turn speeds and reduce open pavement area for pedestrians crossing the street. A consideration of vehicle turning speed and pedestrian crossing distance can contribute to the safe handling of vehicle/pedestrian crossing conflicts (Neuman, 1985). Hauer (1988) noted that "the larger the curb-curve radius, the larger the distance the pedestrian has to cover when crossing the road. Thus, for a sidewalk whose centerline is 1.8 m (6 ft) from the roadway edge, a 4.5-m (15-ft) corner radius increases the crossing distance by only 1 m (3 ft). However, a 15-m (50-ft) radius increases this distance by 8 m (26 ft), or 7 s of additional walking time." Hauer further stated that the following are widely held concerns with the widening of curb radii: (1) the longer the crossing distance, the greater the hazard to pedestrians, even though there may be space for refuge islands when the corner radius is large enough; (2) larger curb radii may induce drivers to negotiate the right turn at a higher speed; and (3) the larger the radius, the wider the turn, which makes it more difficult for the driver and the pedestrian to see each other. For these reasons, the safety of older persons at intersections, particularly pedestrians, may be adversely affected when large curb radii are provided.

In focus group discussions with 46 drivers ages 65–74 (young-old group) and 35 drivers age 75 and older (old-old group), 74 percent of the drivers in each age group reported that tight intersection corner radii posed difficulties maneuvering through the intersection for the following reasons: (1) there are visibility problem with sharp corners; (2) drivers sometimes hit curbs and median barriers; and (3) with sharp turns, trucks turning left into the adjacent opposing traffic lane end up face-to-face with drivers, requiring them to back up (Staplin, Harkey, Lococo, and Tarawneh, 1997). Approximately 24 percent of the young-old drivers and 34 percent of the old-old drivers suggested that medium rounding is sufficient to facilitate turning maneuvers and is safer than very broadly rounded corners because the latter encourages high-speed turns.

In a design preference study using slides to depict varying radii of corner curb cuts, four alternative curb geometries were presented to 30 drivers ages 65–74 (young-old group) and 30 drivers age 75 and older (old-old group) (Staplin et al., 1997). The four alternative geometries

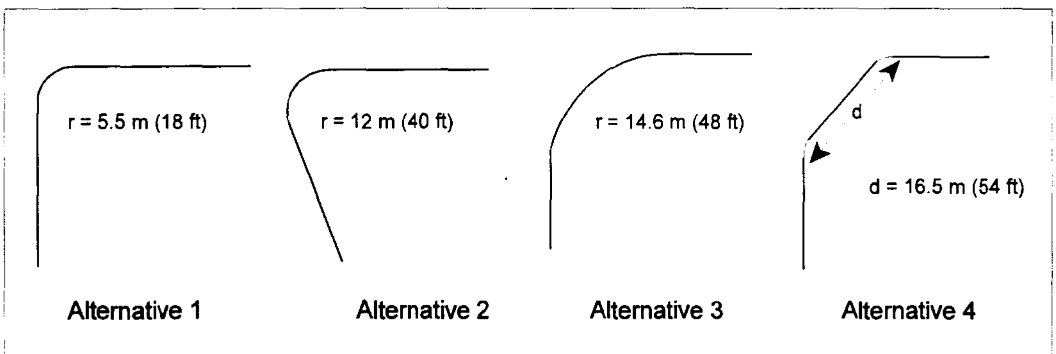

Figure 10. Alternative curb radii evaluated in laboratory preference study conducted by Staplin et al. (1997).

(depicted in figure 10) were: (1) a simple circular radius of 5.5 m (18 ft); (2) a simple circular radius of 12 m (40 ft); (3) a simple circular radius of 14.6 m (48 ft); and (4) a three-sided/truncated curve with the center side measuring 16.5 m (54 ft). The alternatives were identically ranked by both older samples: Alternative 3 was consistently preferred, Alternative 4 placed second, Alternative 2 placed third, and Alternative 1 was least preferred. Both young-old and old-old drivers in this study were most concerned about ease of turning, citing the better maneuverability and less chance of hitting the curb as their primary basis for responding as they did. The second most common-but also strongly weighted-reason for the preferences of both groups related to the degree of visibility of traffic on intersecting roadways, possibly explaining the slight preference for Alternative 2 over Alternative 1. Alternatives 3 and 4 both are described as corner curbline geometries offering ease of turning and good visibility; however, isolated responses to the truncated corner geometry (Alternative 4) indicated concerns that providing *too much* room in the right-turn path might result in a lack of needed guidance information and could lead to a maneuver error, and that it could be harder to detect pedestrians with this design.

In a field study conducted as part of the same project, three intersections providing right-turn curb radii of 12.2 m, 7.6 m, and 4.6 m (40 ft, 25 ft, and 15 ft) were evaluated to examine the effects of curb radii on the turning paths of vehicles driven by drivers in three age groups. One-hundred subjects divided across three age groups drove their own vehicles around test routes using the local street network in Arlington, VA. The three age groups were "young/middle-aged" (ages 25-45), which contained 32 drivers; "young-old" (ages 65-74), which contained 36 drivers; and "old-old" (age 75 and older), which contained 32 drivers. The speed limit was 56 km/h (35 mi/h) and all intersections were located on major or minor arterials within a growing urban area. Data were only collected for turns executed on a green signal phase.

Analysis of the free-flow speeds showed that all factors (age, gender, and geometry), and their interactions, were significant. Mean free-flow speeds were highest at the largest (12.2-m [40-ft]) curb radius location, for all age groups. A consistent finding showed that the slowest mean free-flow speeds were measured at the 4.6-m (15-ft) curb radius location for all age groups. Thus, larger curb radii increased the turning speeds of all drivers, with young/middle-aged and young-old drivers traveling faster than old-old drivers when making right turns.

H. Design Element: Traffic Control for Left-Turn Movements at Signalized Intersections

Table 11. Cross-references of related entries for left-turn movements at signalized intersections.

Applications in Standard Reference Manuals			
MUTCD (2000)	AASHTO *Green Book* (1994)	NCHRP 279, *Intersection Channelization Design Guide* (1985)	*Traffic Engineering Handbook* (1999)
Sect. 1A.13, *approach, intersection, lane-use control signal, regulatory signs, sign legend, & traffic control signal* Sect. 1A.14, *Abbreviations* Table 2B-1 Sects. 2B.17 through 2B.21, 2B.40, 3B.08, & 3B.18 Figs. 3B-11b, 3B-11c, 3B-20, & 3B-21 Sect. 4D.4 Sect. 4D.06 Sects. 4D.07, 4D.08, 4D.12, 4D.15, 4D.16, & 4D.18 Sect. 4J.02 Sects. 4J.03 through 4J.04	p. 319, Para. 2 p. 637, Paras. 6-8 pp. 639-640, Figs. IX-6 & IX-7 p. 641, Para. 1 p. 847, Para. 1 p. 848, Fig. X-17 pp. 852-860, Sect. on *Single-Point Diamond*	p. 1, Item 3, 4th bullet p. 3, 2nd col., Para. 3 p. 21, Fig. 3-1 p. 28, Top fig. p. 29, Top left fig. p. 34, Top fig. & associated notes p. 37, Top left fig. p. 48, Para. 5 & Table 4-3 p. 49, Paras. 1, 2, & 4 and 2nd col, item 2 p. 54, Fig. 4-16, bottom left photo p. 57, Sects. on *Double Left-Turn Lanes--Guidelines for Use & Design of Double Left-Turn Lanes* pp. 58-60, Figs. 4-20 & 4-21 pp. 100-101, Intersect. No. 7 pp. 104-119, Intersect. Nos. 9-16 pp. 132-135, Intersect. Nos. 22-23 p. 144, Intersect. No. 33 pp. 148-151, Intersect. Nos. 35-36	p. 241, Paras. 6 & 9 p. 242, Para. 3 p. 316, Para. 7 pp. 332-333, Sect. on *Storage Lengths* p. 386, Para. 3 p. 427, Para. 3 p. 435, Sect. on *Stop Bars* p. 454, 4th Bullet p. 467, 2nd & 3rd bullets pp. 470-479, Sects. on *Controller Phasing for Left Turns, Operational Modes, & Criteria for Determining Need and Mode* p. 515, Sect. on *Application to Left-Turn Lanes* pp. 522-524, Sect. on *Lane-Use Control Signals*

Crash analyses have shown that older drivers, ages 56–75 and age 76 and older, are overinvolved in left-turn maneuvers at signalized intersections, with failure to yield right-of-way or disregarding the signal the principal violation types (Staplin and Lyles, 1991; Council and Zegeer, 1992). Old-old as drivers (age 75 and older) were more likely than younger drivers (ages 30–50) to be involved in left-turn crashes at urban signalized intersections, and both young-old (ages 65–74) and old-old were more likely to be involved in left-turn crashes at rural signalized intersections. In both cases, the crash-involved older drivers were more likely to be performing a left-turn maneuver than the younger drivers. In addition, Stamatiadis, Taylor, and McKelvey (1991) found that the relative crash involvement ratios for older drivers were higher at two-phase (no turning phase) signalized intersections than for multiphase (including turn arrow) signalized intersections. This highlights problems older drivers may have determining acceptable gaps and maneuvering through traffic streams when there is no protective phase. Furthermore, crash percentages increased significantly for older drivers when an intersection contained flashing controls, as opposed to conventional (red, yellow, green) operations. In this analysis, the greatest crash frequency at signalized intersections occurred on major streets with five lanes, followed closely by roadways containing four lanes. These configurations were most often associated with low-speed, high-volume urban locations, where intersection negotiation requires more complex decisions involving more conflicting vehicles and more visually distracting conditions. Not surprisingly, Garber and Srinivasan's (1991) analysis of 7,000 intersection crashes involving drivers ages 50–64 and age 65 and older found that the provision of a protected left-turn phase will aid in reducing the crash rates of the elderly at signalized intersections.

The change in the angular size of a moving object, such as an approaching vehicle observed by a driver about to turn left at an intersection, provides information crucial to gap judgments (i.e., speed and distance). Age-related declines (possibly exponential) in the ability to detect angular movement have been reported. Older persons may, in fact, require twice the rate of movement than younger persons to perceive that an object is approaching, given a brief duration (2.0 s) of exposure. Also, older persons participating in laboratory studies have been observed to require significantly longer intervals than younger persons to perceive that a vehicle was moving closer at a constant speed: at 31 km/h (19 mi/h), decision times increased 0.5 s between ages 20 and 75 (Hills, 1975).

Compounding this age-related decline in motion perception, some research has indicated that relative to younger subjects, older subjects underestimate approaching vehicle speeds (Hills and Johnson, 1980). Specifically, Scialfa, Guzy, Leibowitz, Garvey, and Tyrrell (1991) showed that older adults tend to overestimate approaching vehicle velocities at lower speeds and underestimate at higher speeds, relative to younger adults. Staplin, Lococo, and Sim (1993), while investigating causes of older driver overinvolvement in turning crashes at intersections, did not find evidence of overestimation of time-to-collision by older drivers in their perception of the closing distance between themselves and another vehicle approaching either head-on or on an intersecting path. However, a relative insensitivity to approach (conflict) vehicle speed was shown for older versus younger drivers, in that younger drivers appropriately adjusted their gap judgment of the "last safe moment" to proceed with a turn in order to take higher approach speeds into account, while older drivers as a group failed to allow a larger gap for a vehicle approaching at 96 km/h (60 mi/h) than for one approaching at 48 km/h (30 mi/h). The interpretation of this and other data in this study was that older drivers rely primarily or exclusively on perceived vehicle separation distance to reach maneuver "go/no go" decisions, reflecting a reduced ability to integrate time and distance information with increasing age. Thus, a principal source of risk at intersections is the error of an older, turning driver in judging gaps in front of fast vehicles.

Aside from [conflict vehicle] motion detection, an additional concern is whether there are age differences in how well drivers understand the rules under which the turns will be made—that is, whether older drivers have disproportionately greater difficulty in understanding the message that is being conveyed by the signal and any ancillary (regulatory) signs. If the signals and markings are not understood, at a minimum, there may be a delay in making a turn or, in the worst case, a crash could result if a protected operation is assumed where it does not exist.

A driver comprehension analysis conducted in a laboratory setting with drivers 30–60 years of age and older showed that green displays (those with the circular green indication alone, green arrow alone, or combinations of circular green and green arrow on the left-turn signal) were correctly interpreted with widely varying frequency, depending on the signals shown for the turning and through movements (Curtis, Opiela, and Guell, 1988). In most cases, performance declined as age increased; older drivers were correct approximately half as often as the youngest drivers. Most driver errors, and especially older driver errors, indicated signal display interpretations that would result in conservative behavior, such as stopping and/or waiting. A summary of the results follows. Overall, green arrows were better understood than circular green indications. Conversely, red and yellow arrows were less comprehensible than circular red and circular yellow indications. Potentially unsafe interpretations were found for red arrow displays in protected-only operations. The yellow arrow display was more often treated as a last chance to complete a turn when compared with a circular yellow indication. Driver errors were most frequent in displays that involved flashing operations; multiple faces with different colors illuminated on the left-turn signal head; and, in particular, different colors on the turn and through signals.

More specifically, Curtis et al. (1988) found that the circular green indication under permitted control was correctly interpreted by approximately 60 percent of the subjects. For protected-only operations, the green arrow (with circular red for through movement) was correctly answered by approximately 75 percent of the drivers. For protected/permitted operation, the circular green alone was correctly answered by only 50 percent of the respondents, while the green arrow in combination with the circular green had approximately 70 percent correct responses. When the circular green with the green arrow was supplemented by the R10-12 sign LEFT TURN YIELD ON GREEN ●, only 34 percent of the drivers answered correctly. This test result suggests that the *MUTCD*-recommended practice may result in some driver confusion, as test subjects answered correctly more often when the sign was *not* present, even when the effects of regional differences in familiarity with the sign were considered. Apparently reinforcing this notion, the Maryland State Highway Administration (MSHA, 1993) reported a higher rate of left-turn collisions at three intersections where the R10-12 sign was installed than at three intersections where the sign was not installed. Unfortunately, driver age was not a study variable; also, medians were present (only) at sites with the R10-12 signs, and differences between sites in terms of signal phasing, traffic volumes and delays, and alignment and other aspects of intersection geometry, though noted, were not described. Other researchers have found improved driver comprehension with the use of the R10-12 sign, compared to other messages informing drivers of the decision rule for protected/permitted operations, as described later in this section.

When Hummer, Montgomery, and Sinha (1990) evaluated motorists' understanding of left-turn signal alternatives, they found that the protected-only signal was by far the best understood, permitted signals were less understood, and the protected/permitted signal the least understood. When a circular green for through traffic and a green arrow for left turns were displayed, the protected signal was clearly preferred over the permitted and protected/permitted signals, and the leading signal sequence was preferred more often than the lagging sequence. Respondents stated that the protected-only signal caused less confusion, was safer, and caused less delay than the permitted and protected/permitted signals. It should be noted, however, that while older persons were in the sample of drivers studied, they made up a very small percentage (8 out of 402) and differences were hard to substantiate.

More recently, Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin (1995) examined the lack of understanding associated with a variety of protected and permitted left-turn signal displays. They found that many drivers, both younger and older, do not understand the protected/permitted signal phasing, and they suggested that efforts to improve motorist comprehension of left-turn signal phasing should be targeted at the entire driving population. In focus group discussions, many older drivers reported that they avoid intersections that do not have a protected-only phase or those where the time allowance for left turns was too short. In addition, the situation where the green arrow eventually turns to a circular green was generally confusing and not appreciated by the older participants. Among the recommendations made by the older drivers were:

- Provide as many protected left-turn opportunities as possible.
- Standardize the sequence for the left-turn green arrow so that it *precedes* solid green or red.
- Lengthen the protected left-turn signal.
- Lengthen the left-turn storage lanes so that turning traffic does not block through traffic.
- Make traffic signal displays more uniform across the United States, including the warning or amber phase.
- Standardize the position and size of signals.
- Provide traffic lights overhead and to the side at major intersections.
- Paint a yellow line in the pavement before the signal in a manner that, if the driver has not reached the line before the light has turned yellow, he/she cannot make it through before the red light.
- Provide borders (backplates) around lights to minimize the effects of glare.
- Eliminate holiday decorations located overhead at intersections, because they are often green and red and may be confusing near signal faces.

Bonneson and McCoy (1994) also found a decreased understanding of protected and permitted left-turn designs with increased age in a survey conducted in Nebraska with 1,610 drivers. In this study, the overlap phase (left-turn green arrow and through circular green illuminated) was the least understood by drivers wishing to turn left, with only one-half of the respondents answering correctly; most of the respondents who erred chose the safer course of action, which was to wait for a gap in oncoming traffic. In terms of signal-head location, 4 to 5 percent more drivers were able to understand the protected/permitted display when it was centered in the left-turn lane (exclusive) as opposed to having the signal head located over the lane line (shared). Although the difference was statistically significant, Bonneson and McCoy point out that the difference may be too small to be of practical significance. In terms of lens arrangement, significantly more drivers understood both the permitted indication and the protected/*MUTCD* indication (left-turn green arrow and through circular red) in vertical and horizontal arrangements than in the cluster arrangement. Comparisons between the protected/*MUTCD* indication and a modified protected indication (green arrow with no circular red) showed that for the horizontal protected/permitted designs, 25 percent more drivers were able to understand the protected indication when the circular red was *not* shown with the green arrow, and for the vertical and cluster protected/permitted designs, 12 percent more drivers understood the modified protected indication. The point is that from an operational perspective, hesitancy as a result of misunderstanding will decrease the level of service and possibly result in crash situations.

An analysis of sign use by Bonneson and McCoy (1994) compared the exclusive cluster lens arrangement over the left-turn lane and exclusive vertical lens arrangement over the through lanes with and without the use of an auxiliary sign (LEFT TURN YIELD ON GREEN ●). Overall, the results indicated that driver understanding of the display increased by about 6 percent when there was *no* sign, though a closer examination of these data revealed that the specific operation signaled by the display was critical. *For the permitted indication, the sign appeared to help driver understanding*, whereas during the overlap and protected indications, it appeared to confuse drivers.

Numerous studies have found that: (1) protected left-turn control is the safest, with protected/permitted being less safe than protected, but safer than permitted (Fambro and Woods, 1981; Matthais and Upchurch, 1985; Curtis et al., 1988); and (2) transitions from protected-only operations to protected/permitted operations experience crash increases (Cottrell and Allen, 1982; Florida Section, Institute of Transportation Engineers, 1982; Cottrell, 1985; Warren, 1985; Agent, 1987). According to Fambro and Woods (1981), for every left-turn crash during a protected phase, 10 would have occurred without protection. Before-and-after studies where intersections were changed from protected to permitted control have shown fourfold to sevenfold increases in left-turn crashes (Florida Section, Institute of Transportation Engineers, 1982; Agent, 1987).

Williams, Ardekani, and Asante (1992) conducted a mail survey of 894 drivers in Texas to assess motorists' understanding of left-turn signal indications and accompanying auxiliary signs. Drivers older than age 65 had the highest percentage of incorrect responses (35 percent). Results of the various analyses are as follows: (1) the use of a green arrow for protected-only left turns produced better comprehension than the use of a circular green indication, even when the circular green indication was accompanied by an auxiliary sign; (2) for a five-section signal head configuration, the display of a green left-turn arrow in isolation produced better driver understanding than the simultaneous display of a circular red indication and a green left-turn arrow; (3) the LEFT TURN YIELD ON GREEN ● auxiliary sign was associated with the smallest percentage of incorrect responses, compared with the LEFT TURN ON GREEN AFTER YIELD sign, the PROTECTED LEFT ON GREEN sign, and the LEFT TURN SIGNAL sign; and (4) the percentage of incorrect responses was 50 percent lower in the presence of a circular red indication compared with a red arrow; the red arrow was often perceived to indicate that a driver may proceed with caution to make a permitted left turn.

In another study conducted by Curtis et al. (1988), it was found that the Delaware flashing red arrow was not correctly interpreted by any subject. The incorrect responses indicated conservative interpretations of the signal displays that would probably be associated with delay and may also be related to rear-end collisions. Drivers interpreted the Delaware signal as requiring a full stop before turning, because a red indication usually means "stop," even though the signal is meant to remind motorists to exercise caution, but not necessarily to stop unless opposing through traffic is present. Hulbert, Beers, and Fowler (1979) found a significant difference in the percentage of drivers younger than age 49 versus those older than age 49 who chose the correct meaning of the red arrow display. Sixty-one percent of the drivers older than age 49 chose "no turning left" compared with 76 percent of those younger than age 49. Although other research has concluded that the left-turn arrow is more effective than the circular red in some left-turn situations, in particular, jurisdictions where special turn signals and exclusive turn lanes are provided (Noel, Gerbig, and Lakew, 1982), drivers of all ages will be better served if signal indications are consistent. Therefore, it is recommended that the use of the arrow be reserved for protected turning movements and the color red be reserved for circular indications that mean "stop."

Hawkins, Womak, and Mounce (1993) surveyed 1,745 drivers in Texas to evaluate driver comprehension of selected traffic control devices. The sample contained 88 drivers who were age 65 and older. Three alternative signs describing the left-turn decision rule were evaluated: (1) R10-9, PROTECTED LEFT ON GREEN ARROW (in the Texas *MUTCD*, but not in the national *MUTCD*); (2) R10-9a, PROTECTED LEFT ON GREEN (in the Texas *MUTCD*, but not in the national *MUTCD*); and (3) R10-12, LEFT TURN YIELD ON GREEN ●. The R10-12 sign did the best job of the signs in the survey of informing the driver of a permitted left-turn condition, with 74.5 percent choosing the desirable response. Of those who responded incorrectly, 13.6 responded that they would wait for the green arrow, and 4.3 percent made the dangerous interpretation that the left turn was protected when the circular green was illuminated. Incorrect responses were more often made by drivers age 65 and older.

The decision-making processes drawing upon working memory crucial to safe performance at intersections may be illustrated through a study of alternative strategies for presentation of left-turn traffic control messages (Staplin and Fisk, 1991). This study evaluated the effect of providing advance left-turn information to drivers who must decide whether or not they have the right-of-way to proceed with a protected turn at an intersection. Younger (mean age of 37) and older (mean age of 71) drivers were tested using slide animation to simulate dynamic approaches to intersection traffic control displays, with and without advance cuing of the "decision rule" (e.g., LEFT TURN MUST YIELD ON GREEN ●) during the intersection approach. Without advance cuing, the decision rule was presented only on a sign mounted on the signal arm across the intersection as per standard practice, and thus was not legible until the driver actually reached the decision point for the turning maneuver. Cuing drivers with advance notice of the decision rule through a redundant upstream posting of sign elements significantly improved both the accuracy and latency of all drivers' decisions for a "go/no go" response upon reaching the intersection, and was of particular benefit to the older test subjects. Presumably, the benefit of upstream "priming" is derived from a reduction in the requirements for serial processing of concurrent information sources (sign message and signal condition) at the instant a maneuver decision must be completed and an action performed.

Stelmach, Goggin, and Garcia-Colera (1987) found that older adults were particularly impaired when preparation was not possible, showing a disproportionate slowing of response when compared with younger subjects. When subjects obtained full information about an upcoming situation, reaction time was faster in all age groups. Stelmach et al. (1987) concluded that older drivers may be particularly disadvantaged when they are required to initiate a movement when there was no opportunity to prepare a response. Preparatory intervals and length of precue viewing times are determining factors in age-related differences in movement preparation and planning (Goggin, Stelmach, and Amrhein, 1989). When preparatory intervals are manipulated in a way that older adults have longer stimulus exposure and longer intervals between stimuli, they profit from the longer inspection times by performing better and exhibiting less slowness of movement (Eisdorfer, 1975; Goggin et al., 1989). Since older drivers benefit from longer exposure to stimuli, Winter (1985) proposed that signs should be spaced farther apart to allow drivers enough time to view information and decide what action to take. Increased viewing time will reduce response uncertainty and decrease older drivers' reaction times.

Differences in maneuver decisions reported by Staplin and Fisk (1991) illustrate both the potential problems older drivers may experience at intersections due to working-memory deficits, and the possibility that such consequences of normal aging can, to some extent, be ameliorated through improved engineering design practices. Staplin and Fisk (1991) also showed that older drivers had higher error rates and increased decision latencies for situations where the left turn was not protected. In particular, the most problematic displays were those with only one steady illuminated signal face (circular green) accompanied by a sign that indicated that it was *not* safe to proceed into the intersection with the assumption of right-of-way (LEFT TURN YIELD ON GREEN ●). A correct response to this combination depends on the inhibition of previously learned "automatic" responses; a signal element with one behavior (go) was incorporated into a traffic control display requiring another conflicting behavior.

Hummer, Montgomery, and Sinha (1991) evaluated leading and lagging signal sequences using a survey of licensed drivers in Indiana, an examination of traffic conflicts, an analysis of crash records, and a simulation model of traffic flow to evaluate motorists' understanding and preference for leading and lagging schemes as well as determining the safety and delay associated with each scheme. Combinations of permitted and protected schemes included: (1) protected-only/leading, in which the protected signal is given to vehicles turning left from a particular street before the circular green is given to the through movement on the same street; (2) protected-only/lagging, in which the green arrow is given to left-turning vehicles after the through movements have been serviced; (3) protected/permitted, in which protected left turns are made in the first part of the phase and a circular green indication allows permitted turns later in the phase; and (4) permitted/protected, in which permitted turns are allowed in the first part of the phase and protected left turns are accommodated later in the phase. The protected-only/leading and protected/permitted schemes are known as "leading," and the protected-only/lagging and permitted/protected are known as "lagging" schemes. Of the 402 valid responses received, 248 respondents preferred the leading sequence, 59 preferred the lagging sequence, and 95 expressed no preference. The most frequent reasons given for preference of the leading sequence were: it is more like normal; it results in less delay; and it is safer. There are apparent tradeoffs here, however; the leading sequence was associated with a higher conflict rate with pedestrians and a higher rate of red-light-running conflicts (drivers turning left during the clearance interval for opposing traffic), while the intersections with a lagging sequence were associated with a significantly higher rate of indecision conflicts than the leading intersections due to violations in driver expectancy. Overall, it is judged that consistency in signal phasing across intersections within a jurisdiction, as well as across jurisdictions, should be a priority, and that use of a leading protected left-turn phase offers the most benefits. A discussion of countermeasures for the protection of pedestrians may be found in the material that presents the Rationale and Supporting Evidence for Design Elements I and P.

Upchurch (1991) compared the relative safety of 5 types of left-turn phasing using Arizona Department of Transportation crash statistics for 523 intersection approaches, where all approaches had a separate left-turn lane, 329 approaches had 2 opposing lanes of traffic, and 194 approaches had 3 opposing lanes. The five types of left-turn phasing included: (1) permitted, (2) leading protected/permitted, (3) lagging protected/permitted, (4) leading protected-only, and (5) lagging protected-only. For the 495 signalized intersections in the State highway system, most samples represented a 4-year crash history (1983–1986). For the 132 signalized intersections in 6 local jurisdictions in Arizona, samples ranged from 4 months to 4 years, all between 1981 and 1989. When the crash statistics were stratified by various ranges of left-turn volume and various ranges of opposing volume (vehicles per day), the following observations and conclusions were made for sample sizes greater than five, eliminating any conclusions about lagging protected-only phasing:

- Leading protected-only phasing had the lowest left-turn crash rate in almost every case. This was true in every left-turn volume range and every opposing volume range except one (19 out of 20 cases). Lagging protected/permitted phasing was the exception for 3 opposing lanes and left-turn volumes of 0 to 1,000.
- When there were two lanes of opposing traffic, lagging protected/permitted phasing tended to have the worst crash rate.
- When there were three lanes of opposing traffic, leading protected/permitted phasing tended to have the worst crash rate.
- When there were two lanes of opposing traffic, the order of safety (crash rate from best to worst) was leading protected-only, permitted, leading protected/permitted, and lagging protected/permitted. However, there was a small difference in the crash rate among the last three types of phasing.
- When there were three lanes of opposing traffic, the order of safety (crash rate from best to worst) was leading protected-only, lagging protected/permitted, permitted, and leading protected/permitted.

Upchurch (1991) compared the crash experience of 194 intersections that had been converted from one type of phasing to another in a simple before-and-after design. For each conversion, 4 years of before-crash data and 4 years of after-crash data were used, where available. At approaches having *two* opposing lanes of traffic, the statistics for conversions from permitted to leading protected/permitted and vice versa reinforced each other, suggesting that leading protected/permitted is safer than permitted. At approaches having *three* opposing lanes of traffic, the statistics for conversions from leading protected-only to leading protected/permitted and vice versa reinforced each other, suggesting that leading protected-only is safer than leading protected/permitted.

Parsonson (1992) stated that a lagging left-turn phase should be used only if the bay provides sufficient storage, since any overflow of the bay during the preceding through movement will spill into the adjacent through lane, blocking it. A lag should also be reserved for those situations in which opposing left-turn movements (or U-turns) are safe from the "left-turn trap" (or are prohibited). The "left-turn trap" occurs when the left-turning driver's right-of-way is terminated, while the opposing (oncoming) approach continues with a green arrow and an adjacent through movement. Thus, left-turning drivers facing a yellow indication are trapped; they believe that the opposing traffic will also have a yellow signal, allowing them to turn on the yellow or immediately after. Since the opposing traffic is not stopping, the turning driver is faced with a potentially hazardous situation. Locations where the left-turn trap is not a hazard include T-intersections, and those where the left turn (or U-turn) opposing the green arrow is prohibited or is allowed only on a green arrow (protected-only phasing). In addition, driver expectancy weighs heavily in favor of leading left turns, and driver confusion over lagging left turns results in losses in start-up time.

I. Design Element: Traffic Control for Right-Turn/RTOR Movements at Signalized Intersections

Table 12. Cross-references of related entries for traffic control for right-turn/RTOR movements at signalized intersections.

Applications in Standard Reference Manuals			
MUTCD (2000)	AASHTO *Green Book* (1994)	NCHRP 279, *Intersection Channelization Design Guide* (1985)	*Traffic Engineering Handbook* (1999)
Sect. 1A.13, *intersection ,right-of-way [assignment], sign legend, & traffic control signal (traffic signal)* Sect. 1A.14, *Abbreviations* Table 2B-1 Sects. 2B.17 through 2B.21, & 2B.40 Sects. 3B.08 & 3B.19 Figs. 3B-11b, 3B-20, & 3B-21 Sects. 4D.04, 4D.05, 4D.07, & 4D.08 Sects. 4D.10 through 4D.12, 4D.15, 4D.16, & 4D.18	p. 319, Para. 2 p. 534, Para. 1 p. 641, Para. 1 p. 718, Paras. 1-2	p. 3, 2nd col, Para. 2 p. 37, Para. 2 & top right fig. pp. 61-65, Sect. on *Exclusive Right-Turn Lanes* p. 100-101, Intersect. No. 7 pp. 106-113, Intersect. Nos. 10-13 pp. 124-125, Intersect. No. 19 pp. 132-135, Intersect. Nos. 22-23 pp. 148-149, Intersect. No. 35	p. 239-242, Sect. on *Turn Restrictions* pp. 332-333, Sect. on *Storage Lengths* p. 384, Item 7 p. 386, Paras. 2 & 6 p. 461, Sect. on *Right-Turn Guidelines for Warrant Application* pp. 522-524, Sect. on *Lane-Use Control Signals*

The right-turn-on-red (RTOR) maneuver provides increased capacity and operational efficiency at a low cost (Institute of Transportation Engineers, 1999). However, traffic control device violations and limited sight distances need to be addressed in order to reduce the potential for safety problems. ITE concluded that a significant proportion of drivers do not make a complete stop before executing an RTOR, and a significant portion of drivers do not yield to pedestrians. In a review of the literature on RTOR laws and motor vehicle crashes, Zador (1984) reported findings that linked RTOR to a 23 percent increase in all right-turning crashes, a 60 percent increase in pedestrian crashes, and a 100 percent increase in bicyclist crashes. Analysis of police crash reports in four States indicated that drivers who are stopped at a red light are looking left for a gap in traffic and do not see pedestrians and bicyclists coming from their right (Preusser, Leaf, DeBartolo, and Levy, 1982). Eldritch (1989) noted that, adding to the adverse effects RTOR has on pedestrian crashes, many motorists persist in making right-turns-on-red even when there is a sign that prohibits the maneuver.

The most recent data available on the safety impact of RTOR were provided by Compton and Milton (1994) in a report to Congress by the National Highway Traffic Safety Administration. Using Fatal Analysis Reporting System (FARS) data and data from four State files for 1989–1992, it was concluded that RTOR crashes represented a small proportion of the total number of traffic crashes in the four States (0.05 percent) and of all fatal (0.03 percent), injury (0.06 percent), and signalized-intersection crashes (0.40 percent). FARS data showed that approximately 84 fatal crashes per year occurred involving a right-turning vehicle at an intersection where RTOR is permitted; however, because the status of the traffic signal indication is not available in this database, the actual number of fatal crashes that occurred when the signal was red is not known. Slightly less than one-half of these crashes involved a pedestrian (44 percent), 10 percent involved a bicyclist, and 33 percent involved one vehicle striking another. Although no data on the age of the drivers involved in RTOR crashes were provided, there are reasons for concern that increasing problems with this maneuver may be observed with the dramatic growth in the number of older drivers in the United States.

The difficulties that older drivers are likely to experience making right turns at intersections are a function of their diminishing gap-judgment abilities, reduced neck/trunk flexibility, attention-sharing deficits, slower acceleration profile, and their general reduction in understanding traffic control devices compared with younger drivers. Right-turning drivers face possible conflicts with pedestrians, and restrictions in the visual attention of older drivers may allow pedestrian and vehicular traffic to go unnoticed. The fact that pedestrians may be crossing the side street, where they enter the path of the right-turning vehicle, places a burden upon the driver to search the right-turning path ahead. The result is the need to share attention between oncoming vehicles approaching from the left and pedestrians in the path to the right. Limitations in the range of visual attention, frequently referred to as "useful field of vision," further contribute to the difficulty of older drivers in detecting the presence of pedestrians or other vehicles near the driver's path. Older drivers, who may have greater difficulty maintaining rapid eye movements and associated head movements, are less likely to make correct judgments on the presence of pedestrians in a crosswalk or on their walking speed (Habib, 1980).

Researchers have identified that the right-turn maneuver is more problematic for older drivers compared with young or middle-aged drivers, presumably as a result of age-related diminished visual, cognitive, and physical capabilities. Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin (1995) conducted an analysis of right-angle, left-turning, right-turning, sideswipe, and rear-end crashes at intersections in Minnesota and Illinois for the time period of 1985–1987, comparing crash proportions and characteristics of "middle-aged" drivers ages 30-50, "young-elderly" drivers ages 65–74, and "old-elderly" drivers age 75 and older. Turning right accounted for 35.8, 39.3, and 42.9 percent, respectively, of the middle-aged, young-elderly, and old-elderly drivers' crashes at urban locations. It appears that for right-turning crashes, the middle-aged driver is most likely crossing the intersection on a green signal and the older drivers are turning right on a red signal in front of the oncoming middle-aged driver. Similar patterns emerged from examination of the rural signalized intersection pre-crash maneuvers, with middle-aged drivers most often traveling straight and older drivers most often turning left or right. Looking at the contributing factors in angle and turning collisions for both rural and urban signalized locations, the middle-aged group was much more likely to be characterized by the police officer as having exhibited "no improper driving." This occurred in 65 percent of the crashes involving this age group, compared with 30.7 percent of the young-elderly and 13.4 percent of the old-elderly. The two elderly groups were more likely to be cited for failing to yield (42.0 percent of the old-elderly, 31.9 percent of the young-elderly, and 10.9 percent of the middle-aged); disregarding the traffic control device (30.7 percent of the old-elderly, 22.0 percent of the young-elderly, and 10.3 percent of the middle-aged); and driver inattention (8.2 percent of the old-elderly, 8.9 percent of the young-elderly, and 6.4 percent of the middle-aged).

Knowledge testing has indicated that, compared with younger drivers, older drivers are less familiar with the meaning of traffic control devices and relatively new traffic laws (McKnight, Simone, and Weidman, 1982). "Newness" of traffic laws, in this regard, relates not to the period of time that has elapsed since the device or law was implemented, but the low frequency with which drivers come in contact with the situation. Older drivers may not encounter right turn on red after stop (RTOR), no turn on red (NTOR), or red right-turn arrow situations on a daily basis, due to the significantly lower amount and frequency of driving in which they are engaged. The demonstrated lack of understanding for the red right-turn arrow (Hulbert, Beers, and Fowler, 1979) and increased violations associated with this display (Owolabi and Noel, 1985) would be of particular concern for older road users drivers and pedestrians alike.

Knoblauch et al. (1995) found that both drivers younger than age 65 and drivers age 65 and older failed to understand that they could turn right on a circular red after stopping in the right lane. Although the survey indicated that older drivers were more likely to stop and remain stopped (45 percent) than younger drivers (36 percent), the differences were not significant.

Staplin, Harkey, Lococo, and Tarawneh (1997) conducted a controlled field study to measure differences in drivers' RTOR behavior as a function of driver age and right-turn lane channelization. In this study, 100 subjects divided across 3 age groups were observed as they drove their own vehicles around test routes using the local street network in Arlington, Virginia. The three age groups were young/middle-aged (ages 25–45), young-old (ages 65–74), and old-old (age 75+). The percentage of drivers who made RTOR maneuvers at the four intersections was included as a measure of mobility.

Staplin et al. (1997) found that significantly fewer drivers in the old-old driver group attempted to make an RTOR (16 percent), compared with young/middle-aged drivers (83 percent) and young-old drivers (45 percent). Similarly, young/middle-aged drivers made an RTOR nearly 80 percent of the time when they had the chance to do so, compared with nearly 36 percent for the young-old drivers and 15 percent for the old-old drivers. Drivers made significantly fewer RTORs at the skewed channelized intersection than at the other three locations. Analysis of the percentage of drivers who made an RTOR without a complete stop showed that age, right-turn lane geometry, gender, and the age-by-geometry interaction were significant. Young/middle-aged drivers made an RTOR without a complete stop nearly 35 percent of the time, compared with nearly 25 percent for the young-old and 3 percent for the old-old drivers. Channelized intersections with or without exclusive acceleration lanes encouraged making an RTOR without a complete stop. The nonchannelized and the skewed locations showed the lowest percentage of RTORs without a complete stop, and were not significantly different from each other. The three age groups showed significantly different performance. Old-old drivers almost always stopped before making an RTOR regardless of the right-turn lane geometry. In only 1 of 26 turns did an older driver *not* stop before making an RTOR; this occurred at the channelized right-turn lane with an acceleration lane. At the nonchannelized intersection (which was controlled by a stop sign), 22 percent of the young/middle-aged drivers, 5 percent of the young-old drivers, and none of the old-old drivers performed an RTOR without a stop. Where an acceleration lane was available, 65 percent of the young/middle-aged drivers continued through without a complete stop, compared with 55 percent of the young-old drivers and 11 percent of the old-old drivers. The increased mobility exhibited by the younger drivers at the channelized right-turn lane locations (controlled by yield signs) was not exhibited by old-old drivers, who stopped in 19 of the 20 turns executed at the channelized locations. In summary, with increases in driver age, the likelihood of RTOR decreases to a very low level for the present cohort of old-old drivers, but when these individuals do engage in this behavior, they are virtually certain to come to a complete stop before initiating the maneuver. Therefore, the emphasis is to ensure adequate sight distance for the older turning driver, to provide sign and signal indications that are most easily understood by this group, and to prompt these motorists to devote adequate attention to pedestrians who may be in conflict with the driver's turning maneuver.

Zegeer and Cynecki (1986) found that offsetting the stop line—moving the stop line of adjacent stopped vehicles back from the intersection by 1.8 to 3.0 m (6 to 10 ft)—was effective in providing better sight distance to the left for RTOR motorists. It also reduced the RTOR conflicts with other traffic and resulted in more RTOR vehicles making a full stop behind the stop line. The offset stop line was recommended as a countermeasure for consideration at RTOR-permitted sites that have two or more lanes on an approach and heavy truck or bus traffic, or unusual geometrics.

Zegeer and Cynecki (1986) also found that a novel sign (circular red symbol with NO TURN ON RED, shown in figure 11) was more effective than the standard black-and-white NO TURN ON RED (R10-11a) sign, especially when implemented near the signal. This countermeasure resulted in an overall reduction in RTOR violations and pedestrian conflicts. The authors suggested that the circular red symbol on the sign helps draw drivers' attention to it, particularly since intersections are associated with a preponderance of signs and information, and they recommended that it should be added to the *MUTCD* as an alternate to, or approved as a replacement of, the current R10-11a design. Increasing the size of the standard NO TURN ON RED sign from its present size of 600 mm x 750 mm (24 in x 30 in) to 750 mm x 900 mm (30 in x 36 in) reduced the proportion of violations at most of the test sites. Finally, Zegeer and Cynecki (1986) found that an electronic NO TURN ON RED blank-out sign was found to be slightly better than the standard *MUTCD* sign in terms of reducing violations, and it was effective in increasing RTOR maneuvers when RTOR was appropriate, thereby reducing vehicle delay. Although the sign is more expensive than standard signs and pavement markings, the authors concluded that it may be justified in situations where pedestrian protection is critical during certain periods (e.g., before and after school) or during a portion of the signal cycle when a separate, opposing left-turn phase may conflict with an unsuspecting RTOR motorist.

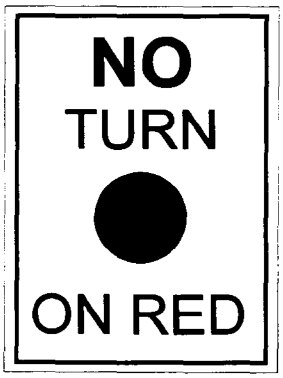

Figure 11. Novel sign tested as a countermeasure by Zegeer and Cynecki (1986).

J. Design Element: Street-Name Signing

Table 13. Cross-references of related entries for street-name signing.

Applications in Standard Reference Manuals	
MUTCD (2000)	AASHTO *Green Book* (1994)
Sect. 1A.14, *Abbreviations*	p. 45, Para. 1
Sects. 2A.08, 2A.12, 2A.15, 2A.17, 2D.01 through 2D.06, 2D.38 & 2E.26	p. 314, Paras. 2-3

The *MUTCD* (1988) states that the lettering on street-name signs (D3) should be at least 100 mm (4 in) high. The *MUTCD* (2000) incorporates a change that specifies that the lettering on post-mounted street-name signs should be at least 150 mm (6 in) high, and that larger letters should be used for street-name signs that are mounted overhead. It provides an option for using 100-mm (4-in) lettering on street-name signs that are posted on local roads with speed limits 40 km/h (25 mi/h) or less. Burnham (1992) noted that the selection of letter size for any sign must evaluate the needs of the user, which are continuously changing as a function of changes in automotive technology, the roadway system, and the population itself. For example, Phoenix, Arizona, a city with a large older driver population, has been using "jumbo" street-name signs at signalized intersections since 1973. These signs are 400 mm (16 in) in height and use 200-mm (8-in) capital letters (*Rural and Urban Roads*, 1973). It is estimated that by the year 2020, 17 percent or more of the population will be older than 65 years of age, and by the year 2030, one in five Americans will be older than age 65 (U.S. Bureau of the Census, 1996). The ability to read street signs is dependent on visual acuity as well as divided attention capabilities, both of which decline significantly with advancing age.

Older drivers participating in focus groups and completing questionnaires for traffic safety researchers over the past decade have consistently stated that larger street signs with bigger lettering and standardization of sign placement overhead would make driving an easier task (Yee, 1985; Gutman and Milstein, 1988; Cooper, 1990; Staplin, Lococo, and Sim, 1990; Benekohal, Resende, Shim, Michaels, and Weeks, 1992; Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin, 1995). Problems with placement included signs that were either obstructed by trees, telephone poles, billboards, or large trucks, or placed too close to or across the intersection rather than on the near side. Older drivers stated that they needed more advance notice regarding upcoming cross-streets and larger street-name signs placed overhead to give them more time to make decisions about where to turn. Also noted were difficulties reading traffic signs with too much information in too small an area, and/or with too small a typeface, which results in the need to slow down or stop to read and respond to the sign's message. May (1992) noted that providing sufficient time to allow motorists to make appropriate turning movements when approaching cross-streets can improve safety and reduce congestion, and that consistent street signing across political jurisdictions can be helpful in this regard, as well as presenting an orderly, predictable picture of the region to tourists, business people, and residents.

Taoka (1991) discussed "spare glance" duration in terms of how drivers allocate their visual search time among different tasks/stimuli. The tasks ranged from side/rearview mirror glances during turning to reading roadway name signs. Although specific results were not differentiated by age, Taoka asserted that 85th percentile glance times at signs (about 2.4 s) were probably too long, since 2.0 s is the maximum that a driver should divert from the basic driving task. Since older drivers are more apt to be those drivers taking the longest time to read signs, these results imply that they will commonly have problems dividing attention between searching for/reading signs and the basic driving task. Malfetti and Winter (1987) observed that older drivers exhibited excessive vehicle-braking behavior whenever a signal or road sign was sighted. This was categorized as an unsafe behavior, because it is confusing and disruptive to following traffic when the lead vehicle brakes for no apparent reason. These researchers obtained many descriptions of older drivers who stopped suddenly at unexpected times and in unexpected places, frequently either within the intersection or 12 m (40 ft) before the intersection to read street signs.

The visibility of retroreflective signs must be considered with regard to their dual requirements of detection and legibility. The sign components affecting detection are sign size, color, shape, brightness, and message or content design. External factors affecting sign detection include its placement (e.g., left, right, or overhead), the visual complexity of the area, and the contrast between the sign and its background. The component parts of retroreflective signs that determine legibility fall into two major classes of variables: character and message. Character variables include the variables related to brightness–i.e., contrast, luminance, color, and contrast orientation–as well as font, letter height, letter width, case, and stroke width. Message variables address the visibility issues of spacing and include interletter, interword, interline, and copy-to-border distances.

Most studies of sign legibility report legibility distance and the letter height of the stimulus; dividing the former measure by the latter defines the "legibility index" (LI), which can serve as a common denominator upon which to compare different studies. Forbes and Holmes (1939) used LI to describe the relative legibility of different letter styles. Under daytime conditions, series B, C, and D were reported to have indexes of 0.4, 0.5, and 0.6 m/mm (33, 42.5, and 50 ft/in), respectively. Forbes, Moskowitz, and Morgan (1950) found the wider series E letters to have an index of 0.66 m/mm (55 ft/in). Over time, the value of 0.6 m/mm (50 ft/in) of letter height became the nominal, though arbitrary and disputed, standard. LI is important to the size requirement determination for a sign in a specific application. Based on the physical attributes of the older driver population, the standard of 0.6 m/mm (50 ft of legibility for every 1 in of letter height), (corresponding to a visual acuity of 20/25) exceeds the visual ability of approximately 40 percent of the drivers between ages 65 and 74. *MUTCD* (2000) section 2A.14, which provides guidance for determining sign letter heights, indicates that sign letter heights should be determined based on 25 mm (1 in) of letter height per 12 m (40 ft) of legibility distance; this shift is certainly desirable considering the human factors issues addressed in this chapter.

Mace (1988), in his work on minimum required visibility distance (MRVD) for highway signs, noted the following relationships:

Required letter size = MRVD / LI *or* Required LI = MRVD / letter size

Either the letter size or the LI may be manipulated to satisfy the MRVD requirement, which specifies the minimum distance at which a sign should be read for proper driver reaction.

Olson and Bernstein (1979) suggested that older drivers should not be expected to achieve an LI of 0.6 m/mm (50 ft/in) under most nighttime circumstances. The data provided by this report gives some expectation that 0.48 m/mm (40 ft/in) is a reasonable goal under most conditions. A 0.48-m/mm (40-ft/in) standard can generally be effective for older drivers, given contrast ratios greater than 5:1 (slightly higher for guide signs) and luminance greater than 10 cd/m^2 for partially reflectorized signs. With regard to the effect of driver age on legibility, Olson, Sivak, and Egan (1983) concluded that older drivers require more contrast between the message and the sign's background than younger drivers to achieve the same level of comprehension. They also noted that legibility losses with age are greater at low levels of background luminance. A reduction in legibility distance of 10 to 20 percent should be assumed when signs are not fully reflectorized. (It should be noted that the *MUTCD* (2000) includes text in section 2A.08 that states that regulatory, warning, and guide signs shall be retroreflective or illuminated to show the same shape and color by both day and night, unless specifically stated otherwise in the *MUTCD* text discussion of a particular sign or group of signs. Section 2D.03 further states that all messages, borders, and legends on guide signs shall be retroreflective, and all backgrounds shall be retroreflective or illuminated. Also, higher surround luminance improved the legibility of signs more for older drivers and reduced the negative effects of excessive contrast. In general, the LI for older drivers is 70 to 77 percent of the LI for younger drivers. The average LI for older drivers is clearly below the nominal value of 0.6 m/mm (50 ft/in) of letter height. The means for older drivers are generally between 0.48 and 0.6 m/mm (40 and 50 ft/in); however, the 85th percentile values reported are between 0.36 and 0.48 m/mm (30 and 40 ft/in) (Sivak, Olson, and Pastalan, 1981; Kuemmel, 1992; Mace, Garvey, and Heckard, 1994). Mace (1988) concluded that a most conservative standard would provide drivers with 2 minutes of arc, which corresponds to 20/40 vision and a 0.36-m/mm (30-ft/in) LI.

In a laboratory simulation study, Staplin et al. (1990) found that older drivers (ages 65–80) demonstrated a need for larger letter sizes to discern a message on a guide sign, compared with a group of younger drivers (ages 19–49). To read a one-word sign, older drivers required a mean letter size corresponding to 2.5 minutes of visual angle (or a Snellen acuity of 20/50), compared with the mean size required by younger drivers of 1.8 minutes of visual angle (or Snellen acuity of 20/35). Character size requirements increased for both age groups when the message contained four words - 3.78 minutes of visual angle (acuity equivalent to 20/75) for the older drivers and 2.7 minutes of visual angle (acuity equivalent to 20/54) for the younger drivers. The main effect of age for the word and message legibility measure was highly significant. Staplin et al. (1990) concluded that for standard highway signing, an increase in character size in the range of 30 percent appears necessary to accommodate age-related acuity differences across the driving population.

Tranchida, Arthur, and Stackhouse (1996) conducted a field study using older drivers who drove the research laboratory's vehicle at night, to determine the legibility distances of street-name signs as a function of sheeting type.

The subjects included nine males ages 68 to 74 and nine females ages 62 to 83. The four sheeting types were: Type IX, Type VII, Type III, and Type I (American Society for Testing and Materials, 2001). Intersections of three levels of complexity were used: high complexity/high traffic activity (e.g., large intersection in downtown business area); intermediate complexity/intermediate traffic activity (e.g., small intersection area in suburban small business/apartment area); and low complexity/low traffic activity (e.g., residential area of single-family homes). All intersections were lighted. Street-name signs with invented names (Strike, Strong, Stress, Straw, Story, and Storm) were created using Series C letters, with a 152-mm (6-in) uppercase "S", followed by 112-mm (4.5-in) lowercase letters. There were no borders on the street-name signs. The signs were placed on the far side of the intersection, either on the right or the left side, and the drivers' task was to read aloud the street name as soon as it was legible to them, as they approached at a speed of 33 km/h (20 mi/h). The vehicle was a two-door sedan with automatic transmission, power steering, and power brakes.

The mean legibility distances across the three intersections and two street sides were as follows for the four sheeting types: Type IX=52.5 m (172 ft); Type VII=51.8 m (170 ft); Type III=43.3 m (142 ft); and Type I=39.6 m (130 ft). Legibility distances were always longer for signs placed on the right side of the street than for those placed on the left. The mean legibility distances for the signs mounted on the right side of the road and corresponding luminances of the sheeting at the legibility distances are as follows: Type IX=61.2 m (201 ft) and 7.369 cd/m^2; Type VII=76.2 m (205 ft) and 4.392 cd/m^2; Type III=53.9 m (177 ft) and 1.1314 cd/m^2; and Type I=53 m (174 ft) and 0.9671 cd/m^2. Sheeting Types VII and IX performed similarly, and produced significantly longer legibility distances than both Type III and Type I sheeting. However, Types VII and Type IX provided significantly longer legibility distances only for the intersections with high-complexity viewing conditions. There was no significant benefit in legibility distance for Type VII and Type IX sheeting at the two streets making up the low-complexity intersection and on one street that was less traveled and less visually complex than the other in the intermediate-complexity intersection.

These results suggest that at visually complex intersections with exaggerated demands for divided attention, the use of retroreflective sheeting that provides increased legibility distance would be of clear benefit to older drivers. Sheeting that provides high retroreflectance overall, particularly at the wide observation angles typical when viewing street-name signs, would best meet this need. The anticipated benefit is that fewer glances will need to be directed toward the sign to determine the legend, and more effort can be devoted to vehicle control and visual search for traffic and pedestrian conflicts.

The use of mixed-case letters on overhead street-name signs was studied by Garvey, Gates, and Pietrucha (1997). Based on this research, it was recommended that for any approach with a 56-km/h (35-mi/h) or lower speed limit, an overhead street-name sign should have 20-cm (8-in) uppercase and 15-cm (6-in) lowercase letters. For approaches with a speed limit above 56 km/h, an overhead street-name sign should contain 25-cm (10-in) uppercase and 20-cm (8-in) lowercase letters. This recommendation is based on the need for street-name signs to be legible for 5.5 s before the intersection, which allows for a 1.5-s alerted perception-reaction time to read a sign and initiate a response (Johannson and Rumar, 1971), plus a 4.0-s interval to complete a combined speed-reduction and tracking task (McGee, Moore, Knapp, and Sanders, 1979). Street-name signs should therefore be readable at 91 m (300 ft) at speeds of 56 km/h (35 mi/h) and at 137 m (450 ft) at 88 km/h (55 mi/h).

In an earlier study, Garvey, Meeker, and Pietrucha (1996) found a 12- to 15-percent increase in recognition distance for mixed-case text over all uppercase legends under both daytime and nighttime conditions. However, this result was for recognition of words that drivers already knew would appear on the signs. Because the reading of street-name signs is often a recognition task rather than a pure legibility task, the reading distance of street-name signs will be higher than would be predicted on driver visual acuity alone. At the same time, street-name legends provide useful information only when they can be read and understood by motorists. This fact underscores the focus on manipulations of those characteristics of sign legends that can increase reading distance. The rationale for mixed-case letters is reported above; the case for enhancements of street-name letter fonts follows. Another obvious manipulation, of course, is simply the size of the letters themselves.

Garvey, Pietrucha, and Meeker (1997) investigated an experimental font in two controlled field studies, using drivers ages 65 to 83. To accurately describe this research, it is necessary to use a trademarked name; however, this does not imply an endorsement of this product by the U.S. Government. Also, until this font undergoes the procedures required for *MUTCD* approval (rule-making process), a recommendation cannot be made to use a non-standard font on standard highway signs. Garvey at al. (1997) compared the recognition distances and legibility distances of words displayed in mixed-case Clearview™ font with those displayed in Standard Highway Series D uppercase font and mixed-case Standard Highway Series E(M) font. The Clearview™ font was developed to have open, wider spaces within a letter, to eliminate the effects of irradiation/halation that is caused by bright, bold stroke widths that "bleed" into a character's open spaces, rendering it illegible. Since each Clearview™ character has more openness than the Standard Highway font, the intercharacter spacing is smaller. Clearview™ spacing results in words that take up 10.8 percent less space than Standard Highway fonts, such that a 12-percent increase in Clearview™ character height results in words equal in sign space to words presented in the Standard Highway fonts. The Clearview™ font was produced in a regular version, with visual proportions similar to the Standard FHWA Series E(M) font, as well as in a condensed version, with visual proportions similar to the Standard FHWA Series D font. Two sizes of the Clearview™ font were displayed: Clearview™ 100 (matched to Standard Highway font height) and Clearview™ 112 (112 percent of Standard Highway font letter height, but equal in overall sign size to Standard Highway font). The fonts tested are described in table 14. The Clearview™ fonts will be referred to as Clear Condensed 100, Clear Condensed 112, Clear 100, and Clear 112 throughout the remainder of this section. White words were created with either encapsulated-lens (ASTM Type III: (R_A=250 cd/lux/m^2) material or microprismatic sheeting designed for short-distance brightness (R_A=430 cd/lux/m^2), and were displayed on a green sign panel measuring 1.2 m^2 (4 ft^2). Each sign contained three place names, each containing six letters (from the same font). The study was conducted using one subject at a time, who was seated in the front passenger's seat of a vehicle driven by the experimenter. For each test run, the vehicle was started at a point 305 m (1000 ft) from the sign.

For the word recognition study, the experimenter read aloud the place name that the subject was to look for on a sign. As the experimenter drove toward the sign at approximately 8 to 16 km/h (5 to 10 mi/h), the subject's task was to tell the experimenter when he or she could determine where the place name was located on the sign: top, middle, or bottom. The distance from the sign at which the subject answered correctly was recorded as the recognition distance. Twelve older drivers (mean age = 70.9 years old) completed the word recognition study during the day, and another 12 older drivers (mean age = 74.8 years old) completed the study at night.

Table 14. Fonts tested by Garvey, Pietrucha, and Meeker (1997).

Font Name	Case	Letter Height
Clear Condensed 100	mixed case	Uppercase: 12.7 cm (5 in) Lowercase: 9.9-cm (3.9-in) loop height
Clear Condensed 112	mixed case	Uppercase: 14.2 cm (5.6 in) Lowercase: 11.2 cm (4.4 in)
Standard Highway Series D	uppercase	12.7 cm (5 in)
Standard Highway Series E(M)	mixed case	Uppercase: 12.7 cm (5 in) Lowercase: 9.9-cm (3.9-in) loop height
Clear 100	mixed case	Uppercase: 12.7 cm (5 in) Lowercase: 9.9-cm (3.9-in) loop height
Clear 112	mixed case	Uppercase: 14.2 cm (5.6 in) Lowercase: 11.2 cm (4.4 in)

A new set of 24 subjects was recruited for the legibility study, with half completing the study during the day (mean age = 71.3 years old) and half at night (mean age = 73.9 years old). For the word legibility study, subjects were presented with only one word on a sign, and were required to read the word. Legibility distance was recorded at the point where subjects correctly read the word.

Results of the word recognition study indicated that during the day, there were no significant differences between either the Clear 100 or Clear 112 and the Series E(M) fonts. However, when comparing the Clear Condensed 100 and Clear Condensed 112 to the Series D font, the mixed-case fonts produced significantly longer recognition distances (14 percent greater) than the all-uppercase Standard Highway font. At night, the Clear 100 font did not produce recognition distances significantly different from those obtained with the Standard Series E(M) font; however, the Clear 112 font produced significantly greater recognition distances (16 percent greater) than the Standard Series E(M) font. The Clear 112 and Clear Condensed 112 fonts produced significantly longer recognition distances than the all-uppercase Series D font. Under both daytime and nighttime conditions, there were no significant effects of material brightness for the word recognition study. The mean daytime and nighttime recognition distances for the six fonts are displayed in table 15.

Table 15. Daytime and nighttime recognition distances for fonts studied by Garvey, Pietrucha, and Meeker (1997).

Font Name	Daytime Recognition Distance	Nighttime Recognition Distance
Clear Condensed 100	120 m (394 ft)	86 m (282 ft)
Clear Condensed 112	134 m (440 ft)	105 m (344 ft)
Standard Highway Series D	117 m (384 ft)	86 m (282 ft)
Clear 100	132 m (433 ft)	103 m (338 ft)
Clear 112	144 m (472 ft)	118 m (387 ft)
Standard Highway Series E(M)	137 m (449 ft)	101 m (331 ft)

The results of the word legibility study conducted during the day indicated that the microprismatic sheeting produced a 4-percent improvement in legibility distance, compared to the encapsulated-lens sheeting. There was no significant interaction between font and material, however. Looking at the effects of font on legibility distance, there was no significant difference in the daytime legibility distances obtained with the Series E(M) font and the Clear 100 and Clear 112 fonts. There was also no significant difference in legibility distance between the Series D font and the Clear 112 and Clear Condensed 112 fonts. However, the all-uppercase Series D font showed significantly longer legibility distances than the Clear Condensed 100 font.

At night, there was a significant interaction effect between font and sheeting material, such that the Clear 112 font produced significantly longer legibility distances (22 percent longer) than the Series E(M) font using the encapsulated-lens sheeting. The microprismatic sheeting showed the same trend (although not significant), with the Clear 112 font producing 11 percent longer legibility distances than the Series E(M). There were no differences between the all-uppercase Series D font and the same-size, mixed-case Clear fonts (i.e., Clear 112 and Clear Condensed 112). However, the Series D font produced significantly longer legibility distances than the Clear Condensed 100 font at night. The legibility distances obtained for the six fonts studied under daytime and nighttime conditions are shown in table 16.

Garvey, Pietrucha, and Meeker (1997) state that guide signs are read using both legibility and recognition criteria, depending on the familiarity of a traveler with the location words used on the signs. A driver who is looking for a particular word on a sign, will be able to read it at a farther distance than a driver who has no idea of what might be on the sign. In the legibility task, the larger letters used with the all-uppercase Series D font produced greater legibility distances than the smaller mixed-case Clear 100 Condensed font. But when the mixed-case font was increased to take up the same sign area as the Series D font (Clear Condensed 112), the legibility distances for the mixed-case and uppercase fonts were the same. But in the recognition task, which Garvey, Pietrucha, and Meeker (1998) state more closely represents real-world behavior, the same-size, mixed-case fonts performed significantly better than the all-uppercase

Series D font. And, even the mixed-case font that took up less sign space performed as well as the all uppercase, Series D font, in terms of word recognition. The authors explain that uppercase words look like blurry rectangles when viewed from a distance. Mixed-case font, on the other hand, produces words with a recognizable overall shape, due to the ascending and descending elements in each letter. The data from this study indicate that if the size of mixed-case words on a sign is matched to the size of words presented in all uppercase font, the mixed-case font provides equal legibility distance and superior recognition distance.

Table 16. Daytime and nighttime legibility distances for fonts studied by Garvey, Pietrucha, and Meeker (1997).

Font Name	Daytime Legibility Distance	Nighttime Legibility Distance
Clear Condensed 100	57 m (187 ft)	45 m (148 ft)
Clear Condensed 112	67 m (220 ft)	59 m (194 ft)
Standard Highway Series D	68 m (223 ft)	63 m (207 ft)
Clear 100	67 m (220 ft)	60 m (197 ft)
Clear 112	70 m (230 ft)	75 m (246 ft)
Standard Highway Series E(M)	68 m (223 ft)	60 m (197 ft)

Next, the *MUTCD* states that street-name signs should be placed at least on diagonally opposite corners so that they will be on the far right-hand side of the intersection for traffic on the major street. Burnham (1992) noted that signs located over the highway are more likely to be seen before those located on either side of the highway. In this regard, Zwahlen (1989) examined detection distances of objects in the peripheral field versus line-of-sight detection and found that average detection distances decrease considerably as the peripheral visual detection angle increases. Placement of street-name signs overhead places the sign in the driver's forward line of sight, eliminating the need for the driver to take his/her eyes away from the driving scene, and reduces the visual complexity of the sign's surround; however, under some sky conditions (e.g., backlit by the sun at dawn and dusk) the sign may be unreadable. Thus, overhead street-name signing should be a supplement to standard roadside signing.

The use of an advance street-name plaque (W16-8) with an advance warning crossroad, side-road, or T-intersection sign (W2-1, W2-2, W2-3, and W2-4) provides the benefit of additional decision-making and maneuver time prior to reaching the intersection. Section 2C.45 of the *MUTCD* (2000) indicates the use of such supplemental street-name signs on intersection warning signs as an option (e.g., an advance street-name plaque *may* be erected separately or below an intersection-related warning sign). The use of advance street-name plaques on advance warning signs has been successful in Phoenix, AZ (*Rural and Urban Roads*, 1973); the size of the lettering on these signs is 200 mm (8 in). Midblock street-name signing provides the same benefit.

Finally, noting Mace's (1988) conclusions supporting a legibility index as conservative as 0.36 m/mm (30 ft/in) to accommodate older drivers, and the practical limitations of increasing sign-panel size, a justification emerges for eliminating the border on street-name signs to permit the use of larger characters. The *MUTCD* (2000) section 2A.15 states that, "Unless specifically stated otherwise, each sign illustrated herein shall have a border of the same color as the legend, at or just inside the edge." In section 2D.38 (Street Name Signs), the *MUTCD* states that, "A border, if used, should be the same color as the legend." The border on street-name signing is presumed to enhance the conspicuity of the sign panel at intersections, where visual complexity and driving task demands may be relatively high. However, the aspect of conspicuity at issue here is "search conspicuity" rather than "attention conspicuity"; as demonstrated by Cole and Hughes (1984), a sign is noticed at a significantly greater distance when a driver expects its presence and knows where to look for it. This is the case with street-name signing at intersections. Detecting the presence of street-name signs isn't the problem—reading them is. Thus, a strong argument can be made that any marginal reduction in conspicuity that may result from eliminating sign borders will be more than offset by the resultant gains in legibility produced by larger characters in the sign legend.

K. Design Element: One-Way/Wrong-Way Signing

Table 17. Cross-references of related entries for one-way/wrong-way signing.

Applications in Standard Reference Manuals		
MUTCD (2000)	AASHTO *Green Book* (1994)	*Traffic Engineering Handbook* (1999)
Sect. 1A.13, *regulatory signs & wrong-way arrows* Sects. 2A.24, 2B.05, 2B.28 through 2B.30, 2B.32, 2B.33, & 2E.50 Figs. 2A-3 through 2A-6 & 2E-31 and 2E-32	P. 519, Paras. 4-5 P. 726, Para. 4 P. 915, Para. 6	P. 384, 1st Principle P. 424, Para. 1 P. 426, Paras. 1-4 P. 436, Sect. on *Word and Symbol Markings* P. 438, Item 4

Vaswani (1974, 1977) found that approximately half of the incidents that involved wrong-way driving on multilane divided highways without access control occurred at intersections with freeway exits and with secondary roads. These wrong-way movements resulted from left-turning vehicles making a left turn into a lane on the near side of the median, rather than turning around the nose of the median into a lane on the far side. In an analysis of 96 crashes resulting from wrong-way movements on divided highways in Indiana from 1970 through 1972, Scifres and Loutzenheiser (1975) found that wrong-way movements most often occur under conditions of low traffic volume, low visibility, and low lane-use density. In addition, it was reported that 69 percent of the wrong-way drivers were drunk, older (age 65 or older), or fatigued (driving between 12 a.m. and 6 a.m.). A review of the literature by Crowley and Seguin (1986) reported that: (1) there are significantly more incidents of wrong-way driving than there are crashes, and (2) drivers older than 60 years of age are overrepresented in wrong-way movements on a per-mile basis.

Further evidence of older driver difficulties likely to result in wrong-way movements was reported by McKnight and Urquijo (1993). These researchers examined 1,000 police forms that documented observations of incompetence when an older driver was either stopped for a violation or involved in a crash. They found that two of the primary behaviors that brought these drivers to the attention of police were driving the wrong way on a one-way street and driving on the wrong side of a two-way street. The drivers' mistakes contributed to many violations (149), but few crashes (29).

The ability to abstract information and make quick decisions are capabilities required to safely perform the driving task. Evidence has been found that older drivers' crashes often occur as the result of overly attending to irrelevant aspects of a driving scene (Planek and Fowler, 1971). Hasher and Zacks (1988) argued that older adults are deficient in inhibitory processes, and as a result, they frequently direct attention to irrelevant information at the expense of relevant information. The selective-attention literature generally suggests that for adults of all ages, but particularly for the elderly, the most relevant information must be signaled in a dramatic manner to ensure that it receives a high priority for processing in situations where there is a great deal of complexity. Mace (1988) stated that age differences in glare sensitivity and restricted peripheral vision, coupled with the process of selective attention, may cause higher conspicuity thresholds for older drivers. Overall, these deficits point to the need for more effective and more conspicuous signs, realized through the provision of multiple or advance signs, as well as changes in size, luminance, or placement of signs.

The most comprehensive survey of current policies and practices for signing intersections to inform drivers of travel direction and to prevent wrong-way movements was conducted by Crowley and Seguin (1986) in the 48 contiguous States and in 35 of the largest cities. They found considerable variability in the location, placement, and types of signs used to prevent wrong-way movements from occurring. The greatest variability in practices was reported in locations where a median divider exists. The study authors reported that median width is a key factor in the number, type, and location of signs to be used. When medians are extremely narrow, there appears to be little confusion that the intersecting roadway is two-way and drivers have less need for special signs to indicate travel direction. Where the median is sufficiently large, the intersection will be generally signed as two separate one-way roadways. A problem in defining what is "wide" and what is "narrow" was shown in the responses from a survey of practitioners across the United States, where there was a significant range in values around the 9 m (30 ft) delineation point specified by the MUTCD (para. 2A-24). The majority of jurisdictions tended to treat wide-median divided highways as if they were two separate intersections for the purpose of direction and turn-prohibition signing. The most commonly reported sign configuration implemented in the jurisdictions that responded to the survey was the MUTCD standard of a pair of ONE WAY signs (R6-1) on the near right-hand corners and far left-hand corners of *each* intersection with the directional roadway. A second pattern reported was a slight variation of the MUTCD standard, where the jurisdictions required a far-right sign (either a ONE WAY or a NO RIGHT TURN symbol sign) at the second intersection. Although many jurisdictions followed the MUTCD specifications for location of signs, many reported that they *replaced* a near-side ONE WAY sign with a NO RIGHT TURN sign (R3-1), even though the MUTCD states that the turn prohibition sign may be used to *supplement* the near-right/far-left pair of ONE WAY signs. The third pattern reported by some jurisdictions was to treat the divided highway, *regardless of median width*, as if it were a single intersection. In this case, a left/median sign for the first one-way roadway and a far-right sign for the second one-way roadway were considered sufficient. Where jurisdictions implement the third pattern, there was more emphasis on the use of the DIVIDED HIGHWAY CROSSING sign (R6-3) to supplement the limited amount of directional information. In one jurisdiction, signing was limited to the use of the DIVIDED HIGHWAY CROSSING sign.

Crowley and Seguin (1986) reported that some jurisdictions recommended the use of optional signs—i.e., DO NOT ENTER (R5-1), WRONG WAY (R5-9), and KEEP RIGHT (R4-7)—but noted that these signs are not helpful to a motorist making decisions as he/she approaches an intersection; they are detected only when the driver begins a wrong-way movement upon reaching the intersection. In this regard, a number of jurisdictions reported that they required the use of the DIVIDED HIGHWAY CROSSING sign, as it is the only sign available that has a direct impact on the decision-making process of drivers approaching a divided highway with a median. The *MUTCD* states that this sign may be used as a supplemental sign on the approach legs of a roadway that intersects with a divided highway. Although this sign was not included in the set of traffic control devices tested by Hulbert and Fowler (1980), these researchers found that where complex driver judgments were required in conjunction with the use and understanding of particular driving situations, larger percentages of drivers failed to correctly respond to the meaning of traffic control devices. The comprehensibility of the DIVIDED HIGHWAY CROSSING sign has not been reliably documented.

Crowley and Seguin (1986) also conducted a laboratory study and a field validation study using subjects in three age groups (younger than age 25, ages 25–54, and age 55 and older) to identify signing practices that best provide information to minimize the possibility of wrong-way turning movements. Subjects were asked to identify driver actions that were, either directly or by implication, prohibited (by signs, markings, etc.), and to do so as quickly as possible. In the laboratory study, projected scenes of intersections containing a median (divided highway) were associated with higher error rates and longer decision-making latencies than scenes containing T-intersections and intersections of a two-way street with a one-way street (no median). The untreated intersections, where geometry alone was tested to determine the extent to which it conveyed an intrinsic "one-way" message, resulted in the worst performance; thus, any signing, regardless of the configuration, appears to be superior to no signing. However, even when the standard *MUTCD* near-right/far-left placement of ONE WAY signs was presented, large numbers of subjects did not recognize that the projected scene was that of a divided highway. Furthermore, the addition of a DIVIDED HIGHWAY CROSSING sign at the near-right corner of the intersection did not significantly reduce the overall error rate. Subjects age 55 and older had fewer correct responses and longer decision-making latencies than subjects in the two younger age groups. Field study results showed the following: (1) unsignalized divided highways resulted in more extreme steering patterns than signalized divided highways at both of the one-way locations; (2) the use of ONE WAY signs in the left/median and far-right locations for medians as narrow as 6 m (20 ft) and as wide as 12.8 m (42 ft) showed superior performance to the single left/median ONE WAY sign; and (3) at undivided intersections of a two-way street with a one-way street, the most extreme variation in steering position was shown for the untreated intersections, suggesting that any signing treatment is better than none.

Crowley and Seguin (1986) noted that because there are intersections with specific physical factors that make the basic near-right/far-left rule inappropriate, the following text should be added to the *MUTCD* (1988) in section 2B-29 (section 2B.32 of the *MUTCD*, 2000) to bring the *MUTCD* and actual practices more in agreement and to reflect the actual manner in which the practitioner must respond to the problem of signing to prevent wrong-way traffic movements while providing positive guidance to drivers: "However, if an engineering study demonstrates the specified placements to be inappropriate due to factors such as sight distance restrictions, approach roadway grade and/or alignment, complex background, etc., one-way signs should be placed so as to provide the best possible guidance for the driver." In addition, a revision to *MUTCD* (1988) section 2A-31 was proposed (section 2A.24 of the *MUTCD*, 2000) which states that for medians of 9 m (30 ft) and less, both the left/median and far-right locations should be implemented when a divided highway justifies any form of one-way signing (see figure 12). DIVIDED HIGHWAY CROSSING, DO NOT ENTER, and WRONG WAY signs are optional, depending on the specific problem at a narrow-median intersection. The authors note, however, that when a median is very narrow, one-way signing is usually unnecessary.

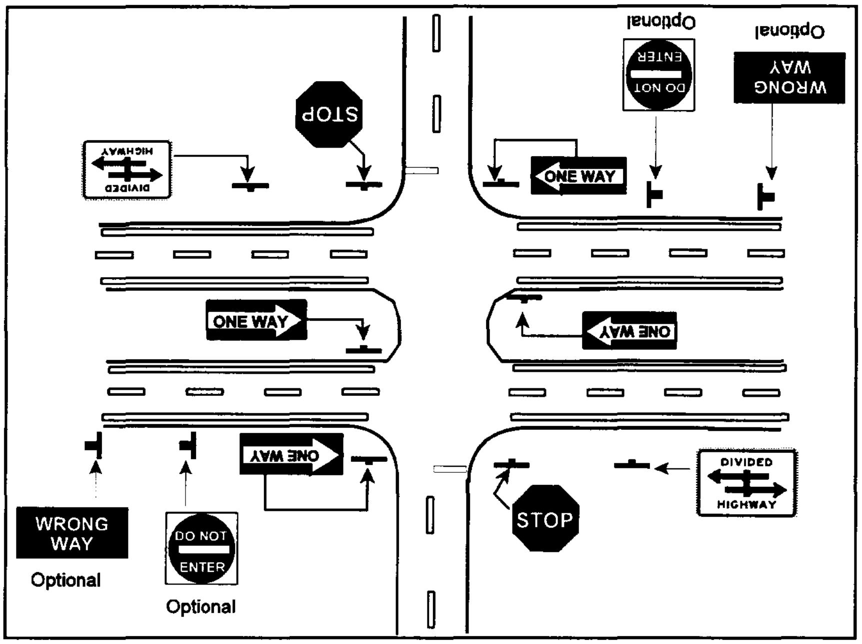

Figure 12. Recommended signing configuration at divided highway crossings for medians less than or equal to 9 m (30 ft), based on evidence provided by Crowley and Seguin (1986).

For medians greater than 9 m (30 ft), Crowley and Seguin (1986) suggested the use of ONE WAY signs posted at each of the following locations for each direction of traffic: near right, median left, and far right. WRONG WAY and DO NOT ENTER signs are again optional. The resulting configuration is consistent with that shown earlier in Recommendation 4 of Design Element E. Because of the large observation and entrance angles for the ONE WAY, KEEP RIGHT, DO NOT ENTER, and WRONG WAY signs, signs using sheeting that provides for high retroreflectivity overall, particularly at wide observation angles and extended entrance angles, are required; otherwise, the signs will be virtually invisible.

For T-intersections, Crowley and Seguin (1986) recommended that near right-side ONE WAY signs and far-side ONE WAY signs be located so that drivers are most likely to see them *before* they begin to make a wrong-way movement. The optimal placement for the far-side sign would be opposite the extended centerline of the approach leg as shown in *MUTCD* (1988) figure 2-4 (figure 2A.6, in the *MUTCD*, 2000). However, where a study indicates that the far-side centerline location is not appropriate at a particular intersection because of blockage, distracting far-side land use, excessively wide approach leg, etc., these authors suggested that the best alternate location is the far left-hand corner for one-way traffic moving from left to right, and the far right-hand corner for traffic moving from right to left (see figure 13).

For four-legged intersections (i.e., the intersection of a one-way street with a two-way street), the near-right/far-left locations were recommended by Crowley and Seguin (1986)

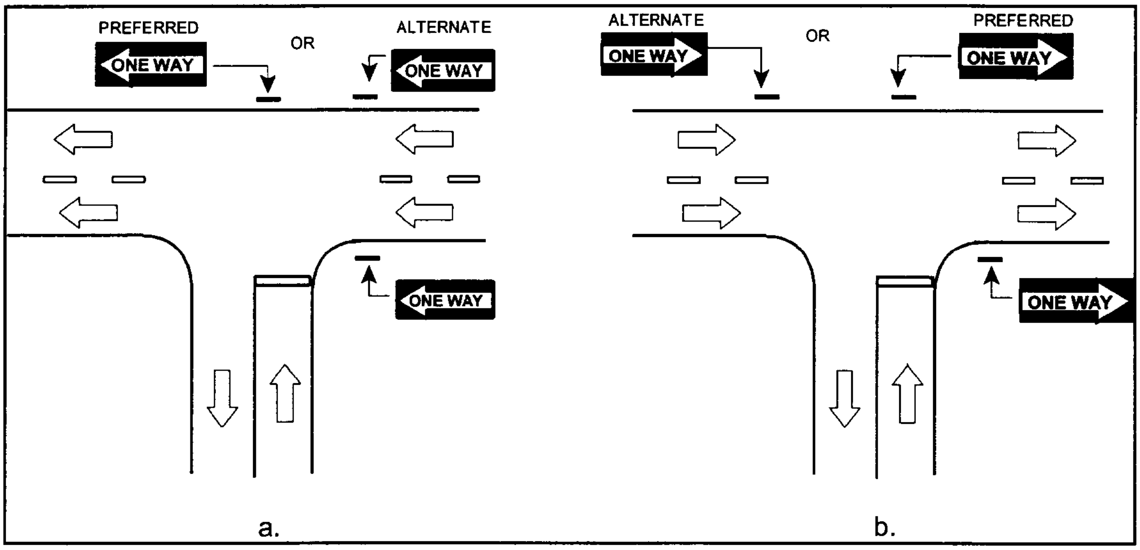

Figure 13. Recommended location of ONE WAY signs for T-type intersections. Source: Crowley and Seguin (1986).

regardless of whether there is left-to-right or right-to-left traffic. An additional ONE WAY sign located on the far-right side may be necessary in certain locations where approach grade and angle may direct the driver's field of view away from the "normal" sign locations (see figure 14).

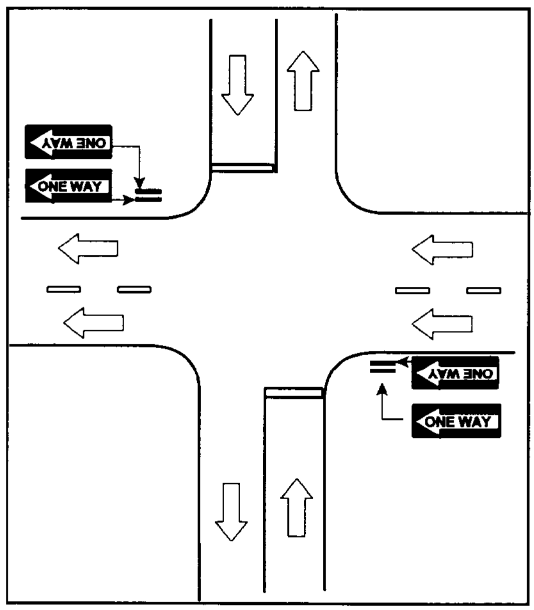

Figure 14. Recommended location of ONE WAY signs for the intersection of one-way and two-way streets. Source: Crowley and Seguin (1986).

Finally, as noted in "Rationale and Supporting Evidence" for Design Element E, the potential for wrong-way movements at intersections with channelized (positive) offset left-turn lanes (within a raised median) increases for the driver turning left from the minor road onto the major road, who must correctly identify the proper median opening into which he/she should turn. The following countermeasures were recommended at intersections with a divided median on the receiving leg, where the left-turn lane treatment results in channelized offset left-turn lanes (e.g., a parallel or tapered left-turn lane between two medians); these countermeasures are intended to reduce the potential for wrong-way maneuvers by drivers turning left from the stop-controlled minor roadway:

- Proper signing (DIVIDED HIGHWAY CROSSING signs and proper positioning of WRONG WAY, DO NOT ENTER, and ONE WAY signing at the intersection) must be implemented.

- The channelized left-turn lanes should contain white lane-use arrow pavement markings (left-turn only).

- Pavement markings that scribe a path through the turn are recommended to reduce the likelihood of a wrong-way movement.

- Placement of 7.1-m- (23.5-ft-) long wrong-way arrows in the through lanes is recommended, as specified in the *MUTCD* (2000) for wrong-way traffic control for locations determined to have a special need (sections 2A.24 and 2E.50). Wrong-way arrows have been shown to reduce the frequency of wrong-way movements at freeway interchanges (Parsonson and Marks, 1979).

- Indistinct medians are considered to be design elements that tend to reduce a driver's ability to see and understand the overall physical and operational features of an intersection, increasing the frequency of wrong-way movements (Scifres and Loutzenheiser, 1975). Delineation of the median noses using reflectorized treatments will increase their visibility and should improve driver understanding of the intersection design and function.

The recommended placement of these traffic control devices was illustrated in Recommendation 4 of Design Element E.

L. Design Element: Stop- and Yield-Controlled Intersection Signing

Table 18. Cross-references of related entries for stop- and yield-controlled intersection signing.

Applications in Standard Reference Manuals			
MUTCD (2000)	AASHTO *Green Book* (1994)	NCHRP 279, *Intersection Channelization Design Guide* (1985)	*Traffic Engineering Handbook* (1999)
Sect. 1A.13, *regulatory signs* Tables 2B-1 & 2C-4 Sects. 2B.03 through 2B.10, 3B.15, & 2C.26	pp. 117-125, Sect. on *Stopping Sight Distance* p. 698, Fig. IX-32B pp. 700-703, Sects. on *Case II-Yield Control for Minor Roads & Case III--Stop Control on Minor Roads* p. 739, Para. 3 p. 919, Sect. on *At-Grade terminals* p. 939, Para. 2	p. 9, Figs. 2-5 & 2-7 p. 10, Table 2-4, 4th bullet p. 21, Fig. 3-1	pp. 235-237, Sects. on *Yield Control & Stop Control* p. 419, Sect. on *Size* pp. 421-423, Sect. on *Types of Retroreflective Sheeting Material* pp. 426, Last para. p. 427, Paras. 1-3 p. 443, Sect. on *Rumble Strips and Speed Humps* pp. 444-445, Sects. on *STOP Sign Warrants, Multiway STOP Warrants, & YIELD Sign Warrants*

Drivers approaching a nonsignalized intersection must be able to detect the presence of the intersection and then detect, recognize, and respond to the intersection traffic control devices present at the intersection. Next, drivers must detect potential conflict vehicles, pedestrian crosswalk locations, and pedestrians at or near the intersection. Proper allocation of attention has become more difficult, as drivers are overloaded with more traffic, more signs, and more complex roadway configurations and traffic patterns, as well as more complex displays and controls in newer vehicles (Dewar, 1992). The presence of large commercial signs near intersections has been associated with a significant increase in crashes at stop-controlled intersections (Holahan, 1977).

Age-related deficits in vision and attention are key to developing recommendations for improved stop control and yield control at intersections. Researchers examining the State crash records of 53 older drivers found that those with restrictions in their "useful field of view," a measure of selective attention and speed of visual processing, had 15 times more intersection crashes than those with normal visual attention (Owsley, Ball, Sloane, Roenker, and Bruni, 1991). A follow-up study with a sample of 300 drivers demonstrated that visual attention deficits could account for up to 30 percent of the variance in intersection crash experience (Ball, Owsley, Sloane, Roenker, and Bruni, 1993). Additional relevant findings may be cited from a simulator study of peripheral visual field loss and driving impairment that also examined the actual driving records of the study participants (Szlyk, Severing, and Fishman, 1991). It was found that visual function factors, including acuity as well as visual field measures, could account for 26 percent of the variance in real-world crashes. Also, greater visual field loss was associated in the simulator data with greater distance traveled ("reaction distance") before responding to a peripheral stimulus (e.g., a stop sign).

A considerable body of evidence exists documenting the difficulties of older driver populations in negotiating stop-controlled intersections. Specifically, analyses of crash and violation types at these sites highlight the older driver's difficulty in detecting, comprehending, and responding to signs within an appropriate time frame for the safe completion of intersection maneuvers.

Statistics on Iowa fatal crashes show that during 1986–1990, running stop signs was a contributing circumstance in 297 fatal crashes that killed 352 people; drivers age 65 and older accounted for 28 percent of the fatal crashes, and drivers younger than age 25 were involved in 27 percent of the fatal crashes (Iowa Department of Transportation, 1991). Stamatiadis, Taylor, and McKelvey (1991) found that at stop-controlled urban intersections, the percentage of drivers age 75 and older involved in right-angle crashes was more than double that of urban signalized intersections. Malfetti and Winter (1987), reporting on the unsafe driving performance of drivers age 55 and older, noted that older drivers frequently failed to respond properly or respond at all to road signs and signals; descriptions of their behavior included running red lights or stop signs and rolling through stop signs. Some older persons' behavior at stop signs and signals seemed to indicate that they did not understand why they needed to wait when no other traffic was coming. Brainan (1980) used in-car observation to gain firsthand knowledge and insight into older people's driving behavior. Drivers in the 70 and older age group showed difficulty at two of the stop signs on the test route; their errors were in failing to make complete stops, poor vehicle positioning at stop signs, and jerky and abrupt stops. Campbell, Wolfe, Blower, Waller, Massie, and Ridella (1990), looking at police reports of crossing crashes at nonsignalized intersections, found that older drivers often stopped and then pulled out in front of oncoming traffic, whereas younger drivers more often failed to stop at all. Further evidence of unsafe behaviors by older drivers was provided in a study by McKnight and Urquijo (1993). Their data consisted of 1,000 police referral forms from the motor vehicle departments of California, Maryland, Massachusetts, Michigan, and Oregon; the forms included observations of incompetent behavior exhibited by older drivers who were stopped for a violation by law enforcement personnel or were involved in a crash. The specific behaviors contributing to the contact between the older driver and the police officer included failing to yield right-of-way or come to a complete stop at a STOP sign, and failing to stop or yield to other traffic; taken together, these behaviors contributed to a significant number of crashes (74) and violations (114).

Data from 124,000 two-vehicle crashes (54,000 crashes at signalized intersections and 70,000 crashes at nonsignalized intersections) showed that drivers younger than age 25 and older than age 65 were overinvolved in crashes at both types of intersections (Stamatiadis et al., 1991). However, the overinvolvement of older drivers in *nonsignalized* intersection crashes was more pronounced than it was for signalized intersection crashes. Although the total number of crashes was reduced at nonsignalized intersections that contained signs when compared with unsigned intersections, the crash involvement ratios of older drivers were higher at signed intersections than at unsigned intersections. At nonsignalized intersections, the highest percentage of fatalities were the result of right-angle collisions (25 percent). In terms of the frequency of injury at nonsignalized intersections, rear-end crashes were the most frequent cause (35 percent), followed by right-angle crashes (18 percent), other-angle crashes (10 percent), and head-on/left-turn crashes (8 percent). The leading violation types for all older drivers in descending order were failure to yield right-of-way, following too closely, improper lane usage, and improper turning. At nonsignalized intersections, older drivers showed the highest crash frequency on major streets with two lanes in both directions (a condition most frequently associated with high-speed, low-volume rural roads), followed by roads with four lanes, and those with five lanes in both directions. These configurations were most often associated with low-speed, high-volume urban locations, where intersection negotiation involves more complex decisions involving more conflict vehicles and more visually distracting conditions.

Cooper (1990) utilized a database of all 1986 police-attended crashes in British Columbia in an effort to compare the crash characteristics of older drivers with those of their younger counterparts. While 66.5 percent of crashes involving drivers ages 36–50 occurred at intersections, the percentage increased to 69.2 percent, 70.7 percent, and 76.0 percent for drivers ages 55–64, 65–74, and 75 and older, respectively. Overall, the two oldest groups identified in this analysis were significantly more involved in crashes at stop/yield sign locations and less involved at either uncontrolled or signal-regulated locations. In follow-on questionnaires administered to a sample of drivers in each age group studied, intersection negotiation was mentioned by the older drivers as second in difficulty to problems changing lanes. About 20 percent of the older drivers mentioned not stopping properly at stop signs. Vehicle maneuvering prior to the crash was a key variable for drivers over age 65, and in particular, for left turns at uncontrolled or stop/yield sign-controlled intersections. Drivers ages 36–50 experienced only 10.9 percent of their crashes while turning left at this type of intersection, compared with 13.0, 15.4, and 19.5 percent of drivers ages 55–64, 65–74, and 75 and older, respectively.

Council and Zegeer (1992) conducted an analysis of intersection crashes occurring in Minnesota and Illinois for the time period of 1985–1987 to highlight crash types, situations, and causes of crashes in an effort to increase the knowledge of how older drivers react at intersections. For all the analyses, comparisons were made between a "young-old" group (ages 65–74), an "old-old" group (age 75 or older), and a "middle-aged" comparison group (ages 30–50). Their findings regarding driver age differences in collision types, pre-crash maneuvers, and contributing factors are described below.

With respect to collision type at stop-controlled intersections, analysis of the data showed little difference in the proportion of crashes involving left-turning vehicles at either urban or rural locations when the older groups were compared with the middle-aged group. There was, however, a significant overinvolvement for both groups of older drivers in right-angle collisions, both in urban and in rural locations. At urban intersections, right-angle collisions accounted for 56.1 percent of the middle-aged driver crashes, compared with 64.7 percent of the young-old and 68.3 percent of the old-old driver crashes. These percentages increase for all groups at rural intersections—61.3, 68.6, and, respectively 71.2 percent for middle-aged drivers, young-old drivers, and old-old drivers. Data for yield-controlled intersections showed older drivers overinvolved in left-turn collisions in urban areas and in angle collisions in both urban and rural areas.

Regarding pre-crash maneuvers at stop-controlled intersections for both rural and urban locations, right-angle collisions were the most frequent collisions, and middle-aged drivers were more likely to be traveling straight or slowing/stopping than the two older groups. The older drivers were more likely to be turning left or starting from a stop than their younger counterparts. The pattern is similar for left-turning crashes. For rear-end collisions, the old-old drivers were more likely to be going straight (thus being the striking vehicle), and the middle-aged and young-old drivers were more likely to be stopped or slowing. For the few right-turning collisions at urban stop-controlled intersections, the middle-aged drivers were going straight and the old-old drivers were more likely to be turning left or right or starting from a stop. Rural stop-controlled locations showed the same patterns of pre-crash maneuvers among the three age groups.

Finally, breakdowns of contributing factors for the urban and rural stop-controlled intersections showed that the middle-aged drivers exhibited a higher proportion of no improper driving behavior, while the young-old and old-old drivers were more often cited for failure-to-yield, disregarding the stop sign, and driver inattention. When starting from a stop, however, the old-old drivers had a *lower* probability of being cited for improper driving. When cited, the old-old group was more likely to have disregarded the stop sign than the other two driver groups. The young-old drivers, as well as the old-old drivers, more frequently failed to yield than the middle-aged drivers.

For left turns, the middle-aged drivers again were more frequently found to have exhibited no improper driving. The two older driver groups were most frequently cited with failure-to-yield. There was no difference in the number of drivers in each age group who disregarded the stop sign. For going-straight situations, the middle-aged driver was found to have exhibited no improper driving behavior twice as often as the young-old driver and almost three times as often as the old-old driver. Failing to yield, disregarding the stop sign, and inattention were most often cited as the contributing factor for the two older groups.

Signing countermeasures to improve safe operation by older drivers at stop- and yield-controlled intersections follow.

Greene, Koppa, Rodriguez, and Wright (1996) noted that the *MUTCD* provides for the possibility of enlarging stop signs where greater emphasis or visibility is required. They proposed an enlargement from the current 750 x 750 mm (30 x 30 in) to 900 x 900 mm (36 x 36 in) at well-traveled intersections or at intersections of small country lanes with State highways. This would also be appropriate at intersections where there is a high incidence of stop-sign-running. Furthermore, Swanson, Dewar, and Kline (1994) reported that older drivers participating in focus group discussions in Calgary, Canada; Boise, Idaho; and San Antonio, Texas indicated a need for bigger and brighter stop signs.

Mace and Pollack (1983) noted that conspicuity is not an observable characteristic of a sign, but rather a construct that relates measures of perceptual performance with measures of background, motivation, and driver uncertainty. In this regard, conspicuity may be aided by multiple treatments or advance signing, as well as changes in size, contrast, and placement. They noted that stop signs following a STOP AHEAD (W3-1a) sign are more conspicuous not only to older drivers, but to everyone, because expectancy has been increased.

The need for appropriate levels of brightness to ensure conspicuity and timely detection of highway signs by drivers, including stop and yield signs, was addressed in FHWA-sponsored research to establish *minimum* retroreflectivity requirements for these devices (minimum maintained levels, as opposed to new or in-service levels). Mace developed a model to derive the retroreflectivity levels necessary for adequate visibility distance, taking into account driver age and visual performance level, as well as the driver's response requirements (action versus no action) to the information presented on a given sign when encountered in a given situation (city, highway) with an assumed operating speed (ranging from 16 km/h [10 mi/h] to 104 km/h [65 mi/h]) for signs of varying size and placement (shoulder, overhead). This work is reported by Ziskind, Mace, Staplin, Sim, and Lococo (1991), who conducted laboratory and controlled field studies using 200 younger and older drivers (ages 16 to 70+) to determine the minimum visibility requirements for traffic control devices. Taking speed and sign application into account, the recommended (minimum maintained level, below which the sign should be replaced) retroreflectivity for stop signs resulting from this research ranged between 10 cd/lux/m^2 up to 24 cd/lux/m^2 for the sign background (red) area, with significantly higher values for the sign legend. For the yield sign, the recommended minimum maintained levels ranged between 24 and 39 cd/lux/m^2. These units (in cd/lux/m^2) or coefficient of retroreflection (R_A) express the efficiency with which the material is able to return incident light at a given geometry between the sign, the vehicle, and the driver. A retroreflectometer is used to obtain these data in the field (reflectivity of a material is measured at specific angles). The observation angle is the angle between the headlamps, the sign, and the driver's eye. The R_A measurements provided by FHWA are all measured at a 0.2-degree observation angle, which corresponds roughly to a viewing distance of 213 m (700 ft), for a right shoulder-mounted sign on a straight road viewed from a passenger sedan. This is important because, in general, as a vehicle approaches a sign, the observation angle becomes larger, reaching 1.0 degrees at 91 m (300 ft), which is roughly legibility distance. Knowing the R_A of a material at 0.2 degrees does not automatically predict its reflectivity at a closer distance (larger observational angle). Because both the stop and yield signs are so extensively overlearned by drivers, their comprehension is believed to be associated with the icon, i.e., their unique shape and coloration. Thus, the brightness of the sign's background area is most critical, because these devices will typically be recognized and understood as soon as they are detected (the conspicuity distance), rather than closer in (legibility distance).

Mercier, Goodspeed, Simmons, and Paniati (1995) conducted a laboratory study using younger and older drivers to measure the minimum luminance thresholds for traffic sign legibility to accommodate varying percentages of the driving population. The purpose of the study was to evaluate the proposed minimum retroreflectivity values derived using CARTS (Computer Analysis of the Retroreflectance of Traffic Signs), which uses a mathematical model to study the relationships between driver variables, vehicle variables, sign variables, and roadway variables (Paniati and Mace, 1993). This model uses MRVD (Minimum Required Visibility Distance), which is the shortest distance at which a sign must be visible to enable a driver to respond safely and appropriately, and includes the distance required for a driver to detect the sign, recognize the message, decide on a proper action, and make the appropriate maneuver before the sign moves out of the driver's view. Paniati and Mace's minimum in-service values (below which sign replacement is indicated) were reported to accommodate an unknown level between 75 and 85 percent of all drivers (see table 19).

The subjects in the Mercier et al. (1995) study included 10 drivers ages 16 to 34; 10 drivers ages 35 to 44; 10 drivers ages 45 to 54; 10 drivers ages 55 to 64; 13 drivers ages 65 to 74; and 10 drivers age 75 or older. All subjects had a visual acuity of at least 20/40. Subjects viewed 25 scaled signs at two distances to simulate minimum required visibility distances (MRVD) traveling at 48 km/h (30 mi/h) and 88 km/h (55 mi/h). Among the signs tested were white-on-red regulatory signs. Illumination levels were manipulated using 20 neutral-density filters ranging from 0.02 to 3.0. Type I engineering-grade sheeting was used for all signs.

Retroreflectance values were calculated based on the luminance levels needed to accommodate 67, 85, and 95 percent of the population of U.S. drivers. Mercier et al. (1995) concluded that the values recommended by Paniati and Mace (1993), reproduced in table 19 for the white-on-red signs, are sufficient to accommodate a high percentage of drivers, with the exception of a few signs, which includes the yield sign. The 95th percentile driver could not be accommodated by the minimum retroreflectivity suggested for the yield sign measuring 76 cm (30 in) for MRVD at both 48 and 88 km/h. The authors point out that increasing brightness for this sign does not increase legibility for older drivers; instead, a redesign of the sign or an enlargement would be needed to enable older drivers to resolve the level of detail required for recognition.

Table 19. Minimum (maintained) retroreflectivity guidelines for white-on-red signs specified by Paniati and Mace (1993) to accommodate 75 to 85 percent of all drivers.

Sign Size	**Speed**	**Minimum Retroreflectivity, cd/lux/m^2 (Paniati and Mace)**
76 cm (30 in)	›72 km/h (≥45 mi/h)	70 (white) 14 (red)
76 cm (30 in)	≤64 km/h (≤40 mi/h)	40 (white) 8 (red)
91 cm (36 in)	›72 km/h (≥45 mi/h)	60 (white) 12 (red)
91 cm (36 in)	≤64 km/h (≤40 mi/h)	35 (white) 7 (red)
122 cm (48 in)	›72 km/h (≥45 mi/h)	50 (white) 10 (red)
122 cm (48 in)	≤64 km/h (≤40 mi/h)	30 (white) 6 (red)

Next, there has been increasing interest in the use of durable fluorescent sheeting for highway signs, because of its increased conspicuity over standard highway sign sheeting, under daytime conditions. Highway signs with fluorescent sheeting have been found to be more conspicuous and can be detected at a farther distance than signs with standard sheeting of the same color. In addition, the color of fluorescent signs is more frequently recognized correctly at farther distances than standard sheeting of the same color (Jenssen, Moen, Brekke, Augdal, and Sjøhaug, 1996; Burns and Pavelka, 1995). Of particular interest, however, are findings reported by Burns and Pavelka (1995) for a field study conducted at dusk (15 min after sunset) without the use of vehicle headlights. In this study, 14 drivers ages 19 to 57 (median age = 40 years) viewed signs with fluorescent red sheeting and signs with standard red sheeting at a distance of 30 m (98 ft). The signs with fluorescent red sheeting were detected by 90 percent of the participants; only 23 percent were able to detect the standard red signs. In terms of correct color recognition, 49 percent were able to correctly recognize the color of the fluorescent red signs at dusk from a distance of 30 m, compared to 12 percent who correctly identified the standard red signs as red. Luminance measurements of the targets and the background were taken for these north-facing signs at dusk, so that luminance contrast ratios could be calculated. The luminance contrast ratio (L_t-L_b/L_b [the luminance of the target minus the luminance of the background, divided by the luminance of the background]) for the fluorescent red signs was 0.7, and for the standard red signs, the luminance contrast ratio was 0.3. The results of this study suggest that the use of fluorescent red sheeting on stop signs would serve to increase their conspicuity, both under daytime and low-luminance conditions, and would be of particular benefit to older drivers who suffer from decreases in contrast sensitivity and have greater difficulty quickly isolating and attending to the most relevant targets in a cluttered visual background. When additional studies quantify the performance gains for older road users, recommendations for relatively widespread use of fluorescent sheeting keyed to specific characteristics of stop- and yield-controlled intersections are likely to emerge. Present recommendations for applications of fluorescent sheeting are limited to the special cases of controlling prohibited movements on freeway ramps (see chapter II) and for passive control systems at highway-rail grade crossings (see chapter V).

A two-way stop requires a driver to cross traffic streams from either direction; this poses a potential risk, because cross-traffic may be proceeding rapidly and drivers may be less prepared to accommodate errors made by crossing or turning drivers. Most critically, drivers proceeding straight through the intersection must be aware of the fact that the cross-street traffic does not stop, and that they must yield to cross-street vehicles from each direction before proceeding through the intersection. Older drivers are disproportionately penalized by the late realization of this operating condition, due to the various sources of response slowing noted earlier. Studies of cross-traffic signing to address this problem have shown qualified, but promising, results in a number of jurisdictions (Gattis, 1996). Although findings indicate that conversion of two-way to four-way stop operations may be more effective in reducing intersection crashes than the use of cross-traffic signing, there are obvious tradeoffs for capacity from this strategy. However, data from crash analyses in Arkansas, Florida, and Oregon reported by Gattis (1996) showed significant reductions in right-angle crashes after cross-traffic signing was installed at problem intersections. Until recently, there was no standard sign design to convey this message. However, Ligon, Carter, and McGee (1985) identified a number of alternate wordings used in different States. In addition, a warrant for use of a cross-traffic sign applied in Illinois may be reviewed in the Gattis (1996) article. The *MUTCD* (2000) indicates in section 2C.27 that a CROSS TRAFFIC DOES NOT STOP plaque (W4-4P) may be used to supplement stop signs on approaches to two-way, stop-controlled intersections where road users frequently misinterpret the intersection as a four-way or all-way stop intersection.

Picha, Schuckel, Parham, and Mai (1996) conducted a survey of 2,129 drivers in five States (California, Minnesota, Mississippi, Pennsylvania, and Texas) to evaluate driver understanding of right-of-way conditions and preference for supplemental signs at two-way, stop-controlled intersections. The majority of the respondents (59 percent) were between ages 25 and 54; however, 22 percent were age 65 or older. The mail survey presented nine supplemental sign designs (three word messages, three symbol messages, and three word-plus-symbol messages), and respondents were asked to choose the preferred sign in each category that best conveyed the right-of-way conditions at a two-way, stop-controlled intersection, and then to choose the most preferred design of the three. The sign most often preferred (by 84 percent of the sample) was the CROSS TRAFFIC DOES NOT STOP word message with a horizontal double-headed arrow symbol. When asked whether a supplemental sign was needed at all two-way, stop-controlled intersections to tell drivers who has the right-of-way (a diagram was provided with the question), 44 percent of the drivers responded "yes," 50 percent "no," and 6 percent "not sure." Picha et al. (1996) provided a list of conditions that may lead a driver to misinterpret an intersection to be all-way stop-controlled, which would justify a supplemental sign treatment. In addition to intersections converted from four-way to two-way stop-control, these include:

- The intersection of two single-jurisdiction roadways (e.g., two State-maintained roadways) in a rural or isolated area.
- Intersections with similar average daily traffic (ADT) volumes on all approaches, but less than the minimum volumes that would warrant the installation of a traffic signal. Typical volumes ranging from 5,000 to 10,000 ADT will probably not meet signal warrants, but could justify a supplemental treatment.
- Intersections with a high conflict frequency and rate, i.e., 20 to 25 conflicts per day (all conflicts combined) or a rate of at least 4 conflicts per 1,000 entering vehicles.
- Intersections with a right-angle crash frequency in the range of three to five (or more) per year. Such a condition may not necessarily meet traffic signal warrants.
- A system of roadway intersections (at-grade) that are not consistent with respect to traffic control schemes.
- Intersections with similar high speeds (i.e., greater than 80 km/h [50 mi/h]) on all approaches.
- Intersections with similar cross-sectional elements (number and width of lanes, shoulders, grades, drainage) on all approaches.

The issue of driver expectancy, a key predictor of performance for older motorists, was addressed in a study by Agent (1979) to determine what treatments would make drivers more aware of a stop-ahead situation. Agent concluded that at rural sites, transverse pavement striping should be applied approximately 366 m (1200 ft) in advance of the stop sign to significantly reduce approach speeds. Later research (Agent, 1988) recommended the following operational improvements at intersections controlled by stop signs: (1) installing additional advance warning signs; (2) modifying warning signs to provide additional notice; (3) adding stop lines to inform motorists of the proper location to stop in order to obtain the maximum available sight distance; (4) installing rumble strips, transverse stripes, or post delineators on the stop approach to warn drivers that they would be required to stop; and (5) installing beacons. Although Agent emphasized that beacons do not eliminate the problem of drivers who disregard the stop sign, flashing beacons used in conjunction with stop signs at isolated intersections or intersections with a restricted sight distance have been consistently shown to be effective in decreasing crashes by increasing driver awareness and decreasing approach speeds (California Department of Public Works, 1967; Cribbins and Walton, 1970; Goldblatt, 1977; King, Abramson, Cohen, and Wilkinson, 1978; Lyles, 1980).

With regard to the crash reduction effectiveness of rumble strips placed on intersection approaches, Harwood (1993) reported that rumble strips can provide a reduction of at least 50 percent in the types of crashes most susceptible to correction, including crashes involving running through a stop sign. They can also be expected to reduce vehicle speed on intersection approaches and to increase driver compliance with stop signs. In an evaluation conducted by the Virginia Department of Highways and Transportation (1981a) where rumble strips were installed at stop-controlled intersections, the total crash frequency was reduced by 37 percent, fatal crashes were reduced by 93 percent, injury crashes were reduced by 37 percent, and property-damage-only crashes were reduced by 25 percent. In this study, 39 of the 141 crashes in the before period were classified as being types susceptible to correction by rumble strip installation, particularly rear-end crashes and ran-stop-sign crashes. The crash rate for these crash types was reduced by 89 percent. Carstens and Woo (1982) found that primary highway intersections where rumble strips were installed experienced a statistically significant reduction in the crash rate in the first year or two following their installation, both at four-way and T-intersections. The crash rate at the 21 study intersections decreased by 51 percent for total crashes and by 38 percent for ran-stop-sign crashes. Carstens and Woo found no statistically significant change in crash rate at 88 intersections on secondary roads where rumble strips were installed. They concluded that rumble strips are more effective on primary highway intersections than on secondary road intersections for the following reasons: (1) primary highways serve a higher proportion of drivers who are unfamiliar with the highway; (2) trips tend to be longer on primary highways so that the fatigue and monotony of driving may play a more important role than on secondary roads; (3) traffic volumes are higher on primary highways, so the number of potential conflicts is greater; and (4) the geometric layout of primary highway intersections is often more complex than that of secondary road intersections. These researchers also found that rumble strips may be more effective in reducing nighttime crashes at unlit intersections than at lit intersections. Harwood (1993) reported that several highway agencies commented that it was important to avoid the temptation to use rumble strips where they are not needed; if every intersection had rumble strips on its approach, rumble strips would soon lose their ability to focus the attention of the motorist on an unexpected hazard.

Before concluding this discussion, certain aspects of yield sign operations deserve mention. A yield sign facilitates traffic flow by preventing unnecessary stops and allowing drivers to enter the traffic flow with minimum disruption of through traffic. Most yield signs are posted where right-turning drivers can approach the cross-street at an oblique angle. Such configurations benefit elderly drivers in carrying out the turning maneuver by avoiding the tight radii that characterize right-angle turns. However, in several respects, intersections regulated by yield signs place greater demands upon drivers than those employing other controls in terms of gap selection, difficulty with head-turning, lane-keeping, and maintaining or adjusting vehicle speed. The angle of approach to the street or highway being entered ranges from the near perpendicular to the near parallel. The closer the angle is to the parallel, the farther the driver must turn his/her head to detect and to judge the speed and distance of vehicles on the road to be entered. Many elderly drivers are unable to turn their heads far enough to get a good look at approaching traffic, while the need to share attention with the road ahead necessarily limits the gap search to 1 or 2 s. Some drivers are reduced to attempting to judge distance and gaps by means of the outside mirror. The inability to judge gaps in this manner often results in the driver reaching the end of the access lane without having identified an appropriate gap. The driver in this situation comes to a complete stop and then must enter the cross-street by accelerating from a stopped position. The difficulty in judging gaps may lead to aborted attempts to enter the roadway, leaving the older driver vulnerable to following drivers who direct their attention upstream and fail to notice that a vehicle has stopped in front of them. The need to share attention between two widely separated points results in eyes being off the intended path for lengthy periods. The diversion of attention, along with movement of the upper torso, hampers the older driver's ability to maintain directional control.

McGee and Blankenship (1989) report that intersections converted from stop to yield control are likely to experience an increase in crashes, especially at higher traffic volumes, at the rate of one additional crash every 2 years. In addition, converted yield-controlled intersections have a higher crash rate than established yield-controlled intersections. They note that while yield control has been found to be as safe as stop control at very low volumes, the safety impacts are not well established for higher volume levels. Agent and Deen (1975) reported that rural road crash types at yield-controlled intersections are different from those at stop-controlled intersections. At yield signs, more than half of the crashes were rear-end collisions, while more than half of the crashes at stop signs were angle collisions.

M. Design Element: Devices for Lane Assignment on Intersection Approach

Table 20. Cross-references of related entries for devices for lane assignment on intersection approach.

Applications in Standard Reference Manuals			
MUTCD (2000)	AASHTO *Green Book* (1994)	NCHRP 279, *Intersection Channelization Design Guide* (1985)	*Traffic Engineering Handbook* (1999)
Sect. 1A.13, *approach* Table 2B-1 Sects. 1A.14 & 2B.19 through 2B.48 Fig. 2B-1 Sects.3A.01, 3A.02, 3A.06, & 3B.05 Figs. 3B-11 Sect. 3B.19 Figs. 3B-19 through 3B-22	pp. 431-432, Sect. on *Width of Roadway* p. 474, Para. 1 p. 517, Para. 5 pp. 629-641, Sects. on *Three-Leg Intersections, Channelized Three-Leg Intersections, Four-Leg Intersections, & Channelized Four-Leg Intersections* p. 740, Paras. 4-5 p. 741, Para. 2 through Table IX-15 on pg. 743 pp. 744-747, Figs. IX-54 through IX-57 pp. 749-751, Sect. on *Speed-Change Lanes at Intersections* pp. 778-792, Sects. on *Continuous Left-Turn Lanes (Two-Way), Auxiliary Lanes, Simultaneous Left Turns, Intersection Design Elements With Frontage Roads, & Bicycles at Intersections*	p. 1, Item 2, 3rd bullet p. 19, Middle fig. p. 21, 2nd col., item 1 p. 24, Para. 1 & top fig. p. 32, Bottom fig. p. 34, Para. 1 & two figs. p. 35, Top-right fig. p. 36, Para. 1 & top and bottom figs. p. 37, Para. 2 & top two figs. pp. 47-48, Sect. on *Warrants/Guidelines for Use of Left-Turn Lanes* p. 51, Fig. 4-12 p. 57, Sects. on *Double Left-Turn Lanes--Guidelines for Use & Guidelines for Implementation of COTWLTL* p. 59, Fig. 4-20 pp. 61-63, Sect. on *Exclusive Right-Turn Lanes* pp. 92-97, Intersect. Nos. 2 & 4 pp. 99-119, Intersect. Nos. 6-16 pp. 132-139, Intersect. Nos. 22-24 & 29 pp. 142-144, Intersect. Nos. 31-33 pp. 146-153, Intersect. Nos. 34-37	p. 241, Sects. on *Pavement Markings, Lane-Use Control Signs, & Multiple Turn Lanes* p. 384, 2nd & 7th principles pp. 429-430, Sect. on *Overhead Signs* p. 434, Sect. on *Transverse Markings* p. 454, 5th bullet pp. 522-524, Sect. on *Lane-Use Control Signals*

As a driver approaches an intersection with the intention of traveling straight through, turning left, or turning right, he/she must first determine whether the currently traveled lane is the proper one for executing the intended maneuver. This understanding of the downstream intersection geometry is accomplished by the driver's visual search and successful detection, recognition, and comprehension of pavement markings (including stripes, symbols, and word markings); regulatory and/or advisory signs mounted overhead, in the median, and/or on the shoulder in advance of the intersection; and other geometric feature cues, such as curb and pavement edge lines, pavement width transitions, and surface texture differences connoting shoulder or median areas. Uncertainty about downstream lane assignment produces hesitancy during the intersection approach; this in, turn, decreases available maneuvering time and diminishes the driver's attentional resources available for effective response to potential traffic conflicts at and near intersections.

Older drivers' decreased contrast sensitivity, reduced useful field of view, increased decision time—particularly in response to unexpected events—and slower vehicle control during movement execution combine to put these highway users at greater crash risk when approaching and negotiating intersections. Contrast sensitivity and visual acuity are the visual/perceptual requirements necessary to detect pavement markings and symbols and to read lane control signs and word and symbol pavement markings. The early detection of lane-control devices, by cuing the driver in advance that designated lanes exist for turning and through maneuvers, promotes safer and more confident performance of any required lane changes. This is because the traffic density is lighter, there are more available gaps, and there are fewer potential conflicts with other vehicles and pedestrians the farther away from the intersection the maneuver is performed. Of course, even the brightest delineation and pavement markings will not be visible to an operator unless an adequate sight distance (determined by horizontal and vertical alignment) is available.

In an effort to analyze the needs and concerns of older drivers, the Illinois Department of Transportation sponsored a statewide survey of 664 drivers, followed up by focus group meetings held in rural and urban areas (Benekohal, Resende, Shim, Michaels, and Weeks, 1992). Within this sample, the following four age categories were used for statistical analyses: ages 66–68, ages 69–72, ages 73–76, and age 77 and older. Comparisons of responses from drivers ages 66–68 and age 77 and older showed that the older group had more difficulty following pavement markings, finding the beginning of the left-turn lane, driving across intersections, and driving during the day. Similarly, the level of difficulty for reading street signs and making left turns at intersections increased with increasing driver age. Turning left at intersections was perceived as a complex driving task, made more difficult when channelization providing visual cues was absent and only pavement markings designated which lane ahead was a through lane and which was a turning lane. The processes of lane location, detection, and selection must be made upstream at a distance where a lane change can be performed safely. Late detection by older drivers will result in erratic maneuvers such as lane weaving close to the intersection (McKnight and Stewart, 1990).

More than half of the 81 older drivers participating in more recent focus group discussions stated that quite often they suddenly find themselves in the wrong lane because: (1) they have certain expectations about lane use derived from intersections encountered earlier on the same roadway, (2) the advance signing is inadequate or lacking, or (3) the pavement markings are covered by cars at the intersection (Staplin, Harkey, Lococo, and Tarawneh, 1997). The biggest problem with turn-only lanes reported by group participants was that there is not enough warning for this feature. The appropriate amount of advance notice, as specified by these drivers, ranged from five car lengths to 1.6 km (1 mi). Sixty-four percent of the participants said that multiple warning signs are necessary when the right lane becomes a turn-only lane, with the need for an initial sign 20 to 30 s away, and a second sign 10 s away from the turn location. The remaining participants said that these distances should be increased.

Even greater consensus was shown in this study regarding sign location for lane assignment. Seventy-nine percent of the group reported that overhead lane-use signs are far more effective than roadside-mounted signs for this type of warning. Several participants suggested that a combination of roadside and overhead signs, in addition to roadway markings, would be beneficial. Although roadway markings were deemed helpful, 84 percent of all participants stated that they are useless in isolation from signs, because they are usually *at* the intersection and are obscured by traffic, and they are frequently worn and faded. The result is that drivers end up in the wrong lane and must go in a direction that they had not planned, or they try to change lanes at a point where it is not safe to do so. Thus, a general conclusion from this study is that overhead signing posted in advance of, as well as at, an intersection provides the most useful information to drivers about movement regulations that may be difficult to obtain from pavement marking arrows when traffic density is high or when pavement markings are obscured by snow or become faded, or where sight distance is limited.

In an early study conducted by Hoffman (1969), the installation of overhead lane-use control signs in advance of six intersections in Michigan contributed to a reduction in the total number of crashes by 44 percent in a 1-year period, and a reduction in the incidence of crashes caused by turning from the wrong lane by 58 percent. More recently, older drivers (as well as their younger counterparts) have been shown to benefit from redundant signing (Staplin and Fisk, 1991). In addition to redundant information about right-of-way movements at intersections, drivers should be forewarned about lane drops, shifts, and merges through advance warning signs, and ideally these conditions should not occur close to an intersection. Advance route or street signing, as well as reassurance (confirmatory) signing/route marker assemblies across the intersection, will aid drivers of all ages in deciding which lane will lead them to their destination prior to reaching the intersection.

The *MUTCD* (2000) specifies in section 2B.18 that Intersection Lane Control signs should be mounted overhead, except where the number of through lanes for an approach is two or less, where the Intersection Lane Control signs (R3-5, R3-6, or R3-8) may be overhead or ground mounted. The Mandatory Movement signs (R3-5, R3-5a, and R3-7) are required to be located where the regulation applies. The Optional Movement Lane Control sign (R3-6) is required to be located at the intersection. The *MUTCD* (2000) section on Advance Intersection Lane Control signs (sign series R3-8, section 2B.21) states that when used, these signs should be placed at an adequate distance in advance of the intersection so that road users can select the appropriate lane (e.g., in advance of the tapers or at the beginning of the turn lane). No guidance is provided regarding overhead versus ground mounting. Section 3B.19 indicates that where through lanes become mandatory turn lanes, signs or markings should be repeated as necessary to prevent entrapment and to help the road user select the most appropriate lane in advance of reaching a queue of waiting vehicles.

Although pavement markings have obvious limitations (e.g., limited durability when installed in areas exposed to heavy traffic, poor visibility on wet roads, and obscuration by snow in some regions), they have the advantage of presenting information to drivers without distracting their attention from the roadway.

Finally, the Institute of Transportation Engineers (ITE) identified several features to enhance the operation of urban arterial trap lanes (through lanes that terminate in an unshadowed mandatory left- or right-turn regulation): (1) signing that gives prominent advance notice of the unexpected mandatory turn regulation, followed by a regulatory sign at the point where the mandatory turn regulation takes effect, followed by a third sign at the intersection itself if there are intervening driveways from which motorists might enter the lane; (2) supplemental pavement markings that consist of a double-width broken lane line beginning at the advance warning sign and extending to the first regulatory sign; (3) a pavement legend in the trap lane; and (4) overhead signing. Candidates for these remediations include left-turn trap lanes on roadways with high volumes, high speeds, poor approach visibility, and complex geometrics (Foxen, 1986).

N. Design Element: Traffic Signals

Table 21. Cross-references of related entries for traffic signals.

Applications in Standard Reference Manuals		
MUTCD (2000)	AASHTO *Green Book* (1994)	*Traffic Engineering Handbook* (1999)
Sect. 1A.13, *flashing (flashing mode) & traffic control signal (traffic signal)* Sects. 4A.02, 4D.01, 4D.04 through 4D.13, 4D.15 through 4D.18, 4E.03 through 4E.07, 4H.02, 4I.02, 4J.03, & 4K.01 MUTCD references to ITE standards ST-008B, ST-011B, ST-010 & Lane-Use Traffic Control Signal Heads	p. 76, Para. 4 p. 79, Para. 3 pp. 318-319, Sect. on *Signals* pp. 480-481, Sect. on *Traffic Control Devices* pp. 534-535, Sect. on *Traffic Control Devices* pp. 716-718, Sect. on *Case IV-Signal Control* p. 637, Paras. 7 & 8 & Fig. IX-7 on p. 640 p. 739, Paras. 4 & 5 p. 847, Para. 1 p. 939, Para. 3	p. 455, 11th bullet p. 496, Sect. on *Visibility and Shielding*

Traffic signals are power-operated signal displays used to regulate or warn traffic. They include displays for intersection control, flashing beacons, lane-directional signals, ramp-metering signals, pedestrian signals, railroad-crossing signals, and similar devices. Warrants for traffic signals are thoroughly described in the *MUTCD*. The decision to install a traffic signal is based on an investigation of physical and traffic flow conditions and data, including traffic volume, approach travel speeds, physical condition diagrams, crash history, and gap and delay information (Wilshire, 1992). The *MUTCD* incorporates the intensity, light distribution, and chromaticity standards from the following Institute of Transportation Engineers (ITE) standards for traffic signals: *Vehicle Control Signal Heads*, ITE Standard No. ST-008B (ITE, 1985b); *Pedestrian Traffic Control Signal Indications*, ITE Standard No. ST-011B (ITE, 1985a); *Traffic Signal Lamps,* ITE Standard No. ST-010 (ITE, 1986); and *Lane-Use Traffic Control Signal Heads* (ITE, 1980). Standards for traffic signals are important because it is imperative that they attract the attention of every driver, including older drivers and those with impaired vision who meet legal requirements, as well as those who are fatigued or distracted, or who are not expecting to encounter a signal at a particular location. It is also necessary for traffic signals to meet motorists' needs under a wide range of conditions, including bright sunlight, nighttime, adverse weather, and visually cluttered surroundings.

To date, studies of traffic signal performance have not typically included observer age as an independent variable. Available evidence suggests, however, that older individuals have reduced levels of sensitivity to intensity and contrast, but not to color. Fisher (1969) reported that as a person ages, the ocular media yellows and has the effect of enhancing the contrast between a red signal and a sky background. However, this effect is more than offset by increasing light scatter within the eye, which diminishes contrast. Older drivers need increased levels of signal luminance and contrast in certain situations to perceive traffic signals as efficiently as 20- to 25-year-old drivers; however, higher signal intensities may cause disabling glare. Fisher and Cole (1974), using data from Blackwell (1970), suggested that older drivers may require 1.5 times the intensity at 50 years of age and 3 times the intensity at 70 years of age, and protanopes (individuals with a color vision deficiency resulting in partial or full insensitivity to red light) may require a fourfold increase. They noted that while increased intensity will ensure that older observers see the signal, the reaction time of older drivers will be longer than for younger drivers. To compensate for this, it would appear necessary to assume a longer required visibility distance, which would result in an increase in the signal intensity required. However, Fisher (1969) also suggested that no increase in signal intensity is likely to compensate for increasing reaction time with age. It should be emphasized that the goal of increased response times for older drivers, requiring longer visibility distances, can also be provided by ensuring that the available signal strength (peak intensity) is maintained through a wide, versus a narrow, viewing angle. This makes signal information more accessible over longer intervals.

It is generally agreed that the visibility issues associated with circular signals relate to the following factors: minimum daytime intensity, intensity distribution, size, nighttime intensity, color of signals, backplates, depreciation (light loss due to lamp wear and dirt on lenses), and phantom illumination (apparent illumination of a signal facing the sun). To place this discussion in context, it should also be noted that traffic signal recommendations for different sizes, colors, and in-service requirements have, in large part, been derived analytically from one research study conducted by Cole and Brown (1966).

In establishing minimum daytime intensity levels for (circular) traffic signals, the two driver characteristics that are considered with regard to the need to adjust peak intensity requirements are color anomalies and driver age. Cole and Brown (1968) determined that the *optimum* red signal intensity is 200 cd for a sky luminance of 10,000 cd/m^2, and an *adequate* signal intensity for this condition would be 100 cd. Cole and Brown (1966, 1968) defined *optimum* as "a signal intensity that provides a very high probability of recognition and which also evokes the shortest response times from the observer." In their research, very high probability was defined as 95 to 100 percent probability of detection. An "adequate signal," although not likely to be missed, results in driver reaction time that is slower than for a signal of "optimum" intensity.

The number of foreign and domestic highway organizations that specify a minimum standard for peak daytime traffic signal intensity is larger than the number of research studies upon which those standards are based. In fact, all of the standards, including those for 200-mm (8-in) and 300-mm (12-in) signals; those for red, yellow, and green signals; and those for new and in-service applications, are derived from a single requirement for a red traffic signal, established from the work of Cole and Brown (1966). The conclusion of this laboratory study was that a red signal with an intensity of 200 cd should invoke a "certain and rapid response" from an observer viewing the signal at distances up to 100 m (328 ft), even under extremely bright ambient conditions. This conclusion was based on experiments in which the background luminance was 5142 cd/m^2. The results were linearly extrapolated to a background luminance of 10,000 cd/m^2, which yielded the 200-cd recommendation. Janoff (1990) concluded that a value of 200 cd minimum intensity for a red signal will suffice for observation distances up to 100 m (328 ft) and vehicle speeds up to 80 km/h (50 mi/h), based on analytic, laboratory, and controlled field experiments performed by Adrian (1963); Boisson and Pages (1964); Rutley, Christie, and Fisher (1965); Jainski and Schmidt-Clausen (1967); Cole and Brown (1968); Fisher (1969); and Fisher and Cole (1974). Fisher and Cole (1974) cautioned against using a value less than 200 cd, to ensure that older drivers and drivers with abnormal color vision will see the signal with certainty and with "reasonable speed."

For green signals, Fisher and Cole (1974) indicated that the ratio of green to red intensity should be 1.33:1, based on laboratory and controlled field research by Adrian (1963), Rutley et al. (1965), Jainski and Schmidt-Clausen (1967), and Fisher (1969), and the ratio of yellow to red should be 3:1, based on research performed by Rutley et al. (1965) and Jainski and Schmidt-Clausen (1967). Janoff (1990) noted that the evidence to support these ratios is somewhat variable, and support of these recommendations is mixed. Table 22, from Janoff (1990), presents the peak intensity requirements of red, green, and yellow traffic signals for 200-mm (8-in) signals for normal-speed roads and for 300-mm (12-in) signals for high-speed roads; the values presented exclude the use of backplates and ignore depreciation. A normal-speed road, in this context, includes speeds up to 80 km/h (50 mi/h), distances up to 100 m (328 ft), and sky luminances up to 10,000 cd/m^2. A high-speed road is defined as one with speeds up to 100 km/h (62 mi/h), distances up to 240 m (787 ft), and sky luminances up to 10,000 cd/m^2. Janoff also noted that although signal size is included, research performed by Cole and Brown (1968) indicated that signal size is not important because traffic signals are point sources rather than area sources and only intensity affects visibility. Thus, the required intensity can be obtained by methods other than increasing signal size (i.e., by using higher intensity sources in 200-mm signals).

Table 22. Peak (minimum) daytime intensity requirement (in cd) for maintained signals with no backplate. Source: Janoff (1990).

Signal Size	Signal Color		
	Red	Green	Yellow
200 mm (8 in)	200	265	600
300 mm (12 in)	895	1190	2685

The specification of standard values for *peak* intensity is important because the distribution of light intensity falls off with increasing horizontal and vertical eccentricity in the viewing angle. Janoff (1990) summarized the peak intensity standards of ITE; Commission Internationale de l'Éclärage (CIE); the British Standards Organization; and standards organizations of Australia, Japan, and South Africa. The US (ITE) standard provides different recommendations for each of the three colors for each signal size. The recommendations are as follows: for red, 157 cd for 200-mm (8-in) signals and 399 cd for 300-mm (12-in) signals; for green, 314 cd for 200-mm (8-in) signals and 798 cd for 300-mm (12-in) signals; and for yellow, 726 cd for 200-mm (8-in) signals and 1848 cd for 300-mm (12-in) signals. Australia recommends the same peak intensity for red and green (200 cd for 200-mm [8-in] signals and 600 cd for 300-mm [12-in] signals), and a yellow intensity equal to three times the red intensity. CIE recommends the same peak intensity for all three colors (200 cd for 200-mm [8-in] signals and 600 cd for 300-mm [12-in] signals), but acknowledges that actual intensity differences between colors result due to the differential transmittance of the colored lenses (1:1.3 for red to green and 1:3 for red to yellow). Japan recommends 240 cd for all three colors. Great Britain recommends a peak intensity of 475 cd for 200-mm (8-in) red and green signals, and 800 cd for 300-mm (12-in) red and green signals. The range for red signals among all of these standards is from 157 cd (ITE) to 475 cd (British Standards Organization). The 157 cd is from research by Cole and Brown. The modal value of 200 cd, specified by Australia, South Africa, and CIE, is based on a depreciation factor of 33 percent.

Only two research reports provide intensity requirements for green and/or yellow signals based on empirical data. Adrian (1963) used a subjective scale and threshold detection criteria in a study that tested red and green signals at different background luminances. He concluded that the intensity requirements for green were 1.0 and 1.2 times that of red for the subjective and threshold studies, respectively. Jainski and Schmidt-Clausen (1967) tested the ability of observers to detect the presence of a red, amber, or green spot, which was either 2 minutes or 1 degree, against varying background luminances. Their results found that green required 1.0 and 2.5 times that of red, and yellow required 2.5 and 3.0 times that of red, for 1 degree and 2 minutes, respectively. Using these results, most standards set requirements for green and yellow to be 1.3 and 3.0 times that of red, respectively. The CIE standard discusses the fact that the ratios of 1.3 and 3.0 for green and yellow appear to reflect the differences in the transmissivity of the varying color lenses.

The most current information on signal intensity requirements that will accommodate road users with age-related vision deficiencies is provided by NCHRP Project 5-15, *Visibility Performance Requirements for Vehicular Traffic Signals*. This investigation includes a series of laboratory and field studies to determine performance-based signal requirements for traffic signal intensity, intensity distribution, and related photometric parameters using a subject population that oversamples older drivers (Freedman, Flicker, Janoff, Schwab, and Staplin, 1997). While the final results and recommendations from this research were not yet published when this Handbook was prepared, one preliminary finding deserves emphasis: minimum daytime brightness requirements must be stated in terms of *maintained* signal performance levels. The present recommendation in this area accordingly augments the 200-cd intensity requirement for red 200-mm (8-in) signals that appears most prominently in the literature cited above (e.g., Janoff, 1990) with this emphasis on in-service performance measurement.

Holowachuk, Leung, and Lakowski (1993) conducted a laboratory study to evaluate the effects of color vision deficiencies and age-related diminished visual capability on the visibility of traffic signals. Subjects ranged in age from 18 to 80 and older, and included 64 individuals with normal color vision and 51 subjects who were color vision deficient. A laboratory simulation apparatus was used to present photographs taken of seven signal-head assemblies at intersections at distances of 50 and 100 m (164 and 328 ft). The photographs were taken at intersections in the Vancouver area within simple and complex environments. Each subject viewed 48 photographs shot during daylight conditions and 38 photographs shot at night. Subjects' reaction times to recognize the color of the "on" signal were measured, as was the accuracy of the response. The basic highway signal head used by the Ministry of Transportation and Highways in British Columbia consists of a 300-mm (12-in) red light, a 200-mm (8-in) amber light, and a 200-mm (8-

in) green light arranged vertically with a yellow backplate. This "standard highway" signal plus six other off-the-shelf signal-head designs were used in the study, as shown in table 23.

Results indicated that color vision deficient drivers had significantly longer reaction times than drivers with normal color vision, and older drivers had longer reaction times compared to younger drivers. Of particular importance is that the reaction times of the normal color vision drivers over age 50 (n=15) compared closely to those of color vision deficient drivers (n=50). Regarding signal design, for daytime conditions, the No Backplate assembly produced the longest reaction times for both the normal color vision and the color vision deficient drivers. Reaction times for the larger and brighter lenses (shape-coded and 300 RYG [red, yellow, green]) were the shortest for both groups of subjects. For nighttime conditions, the signal assemblies showed few differences in reaction time for subjects with normal color vision. Reaction times were shortest for the shape-coded and 300 RYG assemblies; however, the baseline assembly and the No Backplate assemblies produced the longest reaction times. For the color vision deficient group, the reaction times for the shape-coded, 300 RYG, and the Modified Backplate assemblies were distinctly shorter than those for the Baseline and No Backplate assemblies. Nighttime reaction times were much longer than daytime reaction times for the subjects with color vision deficiencies. Signal-light colors were identified more incorrectly for nighttime conditions than for daytime conditions. This difference was greatest for the older color vision deficient drivers (n=22).

Table 23. Signal-head designs evaluated by Holowachuk et al. (1993, 1994).

Name	Abbreviation	Lens Size (mm)*	Backplate	Other Features
No Backplate	N BP	Red 200, Amber 200, Green 200	No	N/A**
Base Line	Baseline	Red 200, Amber 200, Green 200	Yes	N/A
Modified Backplate	Mod BP	Red 200, Amber 200, Green 200	Yes	Backplate with 50-mm reflective border
Standard Highway	Std Hwy	Red 300, Amber 200, Green 200	Yes	N/A
300-mm LED	LED	Red 300 (LED), Amber 200, Green 200	Yes	300-mm red LED signal
300-mm Red, Amber, Green	300 RYG	Red 300, Amber 300, Green 300	Yes	N/A
300-mm Shape-Coded	Shape-Coded	Red 300, Amber 300, Green 300	Yes	Red square Amber diamond Green circle

* 300-mm lens uses 150-W bulb; 200-mm lens uses 69-W bulb.
** N/A-Not applicable.

Overall, findings indicated that the reaction times for all subjects were the shortest for signal designs with larger lenses (300 mm) and higher luminances (150-W bulbs). There was no significant difference in reaction times between the shape-coded and the 300 RYG assemblies for the normal subjects or for the color vision deficient subjects. The next-best performing signal design was the Modified Backplate. The signal assembly with no backplate produced the longest reaction times. Based on these findings, a new signal specification was established for field testing, consisting of all 300-mm signal lenses and a backplate with an additional 75 mm of reflective border. This new assembly has been under test since 1994 at 10 treatment and 10 control intersections located on major highway corridors in Burnaby, Maple Ridge, Surrey, and Saanich (British Columbia, Canada). In a preliminary evaluation, the total number of collisions was reduced by 24 percent as a result of the new signal-head design, and the severity was reduced by 20 percent (Leung, in progress).

Some research has indicated that the dimming of signals at night may have advantages, while also reducing power consumption. Freedman, Davit, Staplin, and Breton (1985) conducted a laboratory study and controlled and observational field studies to determine the operational, safety, and economic impact of dimming traffic signals at night. Results indicated that drivers behaved safely and efficiently when signals were dimmed to as low as 30 percent of ITE recommendations. Previously, however, Lunenfeld (1977) cited the considerable range of night background luminances that may occur in concluding that in some brightly lit urban conditions, or where there is considerable visual noise, daytime signal brightness is needed to maintain an acceptable contrast ratio. The ITE standard presently does not differentiate between daytime and nighttime intensity requirements. CIE recommends that intensities greater than 200 cd or less than 25 cd be avoided at night and advises a range of 50 to 100 cd for nighttime, except for high-speed roads where the daytime values are preferred. The South African and Australian standards allow for dimming, but do not recommend an intensity level. While the option for dimming on a location-by-location basis should not be excluded, from the standpoint of older driver needs, there is no compelling reason to recommend widespread reduction of traffic signal intensity during nighttime operations.

It is common practice to try to enhance the visibility of signals by placing a large, black surround behind the signals. The backplate, rather than the sky, becomes the background of the signals, enhancing the contrast. Regarding backplate size, no recommendation is contained in the ITE standard. CIE (1988), however, recommends that all signals use backplates of a size (width) of three times the diameter of the signal. As a practical matter, the use of a backplate also serves to compensate, in part, for the effects of depreciation, since a backplate reduces the required intensity by roughly 25 percent (Cole and Brown, 1966), while depreciation increases the requirement by the same amount. Guidelines published by CIE (1988) include an allowance of 25-percent transmissivity for depreciation due to dirt and aging (a 33-percent increase in intensity for new installations). The 200-cd requirement for red signals, as noted earlier, must be met *after* the depreciation factor has been taken into account.

Regarding signal size, section 4D.15 of the *MUTCD* specifies that the two nominal diameter sizes for vehicular signal lenses are 200 mm (8 in) and 300 mm (12 in), and provides guidance that states that 300-mm (12-in) lenses should be used at locations where there is a significant percentage of older drivers. Researchers at the Texas Transportation Institute propose that the larger 300-mm (12-in) lens should be used to improve the attention-getting value of signals for older drivers (Greene, Koppa, Rodriguez, and Wright, 1996). Use of the large lens also provides motorists with more time to determine the signal color and to make the correct response.

A final issue with respect to signal performance and older drivers is the change intervals between phases, and the assumptions about perception-reaction time (PRT) on which these calculations are based. At present, a value of 1.0 s is assumed to compute change intervals for traffic signals, a value which, according to Tarawneh (1991), dates back to a 1934 Massachusetts Institute of Technology study on brake-reaction time. Tarawneh examined findings published by proponents of both "parallel" and "sequential" (serial) models of driver information processing, seeking to determine the best estimator for older individuals of a PRT encompassing six different component processing operations: (1) latency time (onset of stimulus to beginning of eye movement toward signal); (2) eye/head movement time to fixate on the signal; (3) fixation time to get enough information to identify the stimulus; (4) recognition time (interpret signal display in terms of possible courses of action); (5) decision time to select the best response in the situation; and (6) limb movement time to accomplish the appropriate steering and brake/accelerator movements.

Tarawneh's (1991) review produced several conclusions. First, the situation of a signal change at an intersection is among the most extreme, in terms of both the information-processing demand and subjective feelings of stress that will be experienced by many older drivers. Second, the most reasonable interpretation of research to date indicates that the best "mental model" to describe and predict how drivers respond in this context includes a mix of concurrent and serial-and-contingent information-processing operations. In this approach, the most valid PRT estimator will fall between the bounds of values derived from the competing models thus far, also taking into account age-related response slowing for recognition, decision-making, and limb movement. After a tabular summary of the specific component values upon which he based his calculations, Tarawneh (1991) called for an increase in the current PRT value used to calculate the length of the yellow interval (derived from tests of much younger subjects) from 1.0 s to 1.5 s to accommodate older drivers.

A contrasting set of results was obtained in a recent FHWA-sponsored study of traffic operations control for older drivers (Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin, 1995). This study compared the decision/response times and deceleration characteristics of older drivers (ages 60–71 and older) with those of younger drivers (younger than age 60) at the onset of the amber signal phase. Testing was conducted using a controlled field test facility, where subjects drove their own vehicles. Subjects were asked to maintain speeds of 48 km/h (30 mi/h) and 32 km/h (20 mi/h) for certain test circuits. The duration of the amber signal was 3.0 s before turning to red. In half of the trials, the signal changed from green to amber when the subject was 3.0 to 3.9 s from the signal, and in the remaining trials, when the subject was 4.0 to 4.9 s away from the signal. For three of the circuits, subjects were asked to brake as they normally would and to stop before reaching the intersection, if they chose to do so. During a fourth circuit, they were asked to brake to a stop, if they possibly could, if the light changed from green to amber. Response times were measured for the drivers who stopped, from the onset of the amber phase to the time the brake was applied.

Results of the Knoblauch et al. (1995) study showed no significant differences in 85th percentile decision/response times between younger and older drivers when subjects were close to the signal at either approach speed. The 85th percentile decision time of younger subjects was 0.39 s at 32 km/h (20 mi/h) and 0.45 s at 48 km/h (30 mi/h). For older drivers, these times were 0.51 and 0.53 s for 32 km/h and 48 km/h (20 mi/h and 30 mi/h), respectively. When subjects were farther from the signal at the amber onset, older drivers had significantly longer decision/response times (1.38 s at 32 km/h [20 mi/h] and 0.88 s at 48 km/h [30 mi/h]) than the younger drivers (0.50 s at 32 km/h [20 mi/h] and 0.46 s at 48 km/h [30 mi/h]). The authors suggested that the significant differences between older and younger drivers occurred when the subjects were relatively far from the signal, and that some older subjects will take longer to react and respond when additional time is available for them to do so. Thus, they concluded that the older drivers were not necessarily reacting inappropriately to the signal. In terms of deceleration rates, there were no significant differences, either in the mean or 15th percentile values, between the older and younger subjects. Together, these findings led the authors to conclude that no changes in amber signal phase timing are required to accommodate older drivers.

Taking the review and study findings of Tarawneh (1991) and Knoblauch et al. (1995) into consideration, an approach that retains the 1.0-s PRT value as a minimum for calculating the yellow change interval seems appropriate; however, to acknowledge the significant body of work documenting age-related increases in PRT, the use of a 1.5-s PRT is well justified when engineering judgment determines a special need to take older drivers' diminished capabilities into account. A recommendation for an all-red clearance interval logically follows, with length determined according to the ITE (1992).

O. Design Element: Fixed Lighting Installations

Table 24. Cross-references of related entries for fixed lighting installations.

Applications in Standard Reference Manuals		
MUTCD (2000)	AASHTO *Green Book* (1994)	*Roadway Lighting Handbook* (1978)
Sect. 1A.13, *sign illumination* Sects. 2A.08, 2E.05, & 3G.04 Sect. 4B.04 Sects. 6D.01 & 6D.02 Sect. 6F.70 Sect. 6G.13 Table 6H-2 Figs. 6H-12, 6H-40, & 6H-41 Sects. 8C.01 & 10C.14	pp. 309-311, Sect. on *Lighting* p. 315, Para. 2 pp. 440-441, Sect. on *Street and Roadway Lighting* p. 480, Sect. on *Street and Roadway Lighting* pp. 545-546, Sect. on *Lighting* p. 567, Para. 1 & Fig. VIII-6 on p. 570 p. 792, Sect. on *Lighting at Intersections*	pp. 16-27, Sects. on *Analytical Approach to Illumination Warrants, Informational Needs Approach to Warrants, & Warrants for Rural Intersection Lighting* pp. 29-30, Sect. on *Adverse Geometry and Environment Warrant* pp. 42-45, Sect. on *Summary of Light Sources* pp. 53-56, Sect. on *Classification of Luminaire Light Distributions* p. 71, 5th bullet p. 94, Sect. on *Coordination of the Arterial Lighting System and Traffic Controls* p. 96, Sect. on *Intersection Lighting* pp. 98-99, Sect. on *Rural Intersection Lighting* pp. 120-129, Sect. on *Illumination Design Procedure* pp. 187-200, Sect. on *Maintaining the System*

One of the main purposes of lighting a roadway at night is to increase the visibility of the roadway and its immediate environment, thereby permitting the driver to maneuver more safely and efficiently. The visibility of an object is that property which makes it discernible from its surroundings. This property depends on a combination of factors; principally, these factors include the differences in luminance, hue, and saturation between the object and its immediate background (contrast); the angular size of the object at the eye of the observer; the luminance of the background against which it is seen; and the duration of observation.

Of all the highway safety improvement projects evaluated by FHWA (1996), using data from 1974 to 1995 where before-and-after exposure data were available, intersection illumination was associated with the highest benefit-cost ratio (26.8) in reducing fatal and injury crashes. The link between reduced visibility and highway safety is conceptually straightforward. Low luminance contributes to a reduction in visual capabilities such as acuity, distance judgment, speed of seeing, color discrimination, and glare tolerance, which are already diminished capabilities in older drivers.

The CIE (1990) reports that road crashes at night are disproportionately higher in number and severity compared with crashes during the day. Data from 13 Organisation for Economic Co-operation and Development countries showed that the proportion of fatal nighttime crashes ranged between 25 and 59 percent (average value of 48.5 percent). In this evaluation of 62 lighting and crash studies, 85 percent of the results showed lighting to be beneficial, with approximately one-third of the results being statistically significant.

In 1990, drivers (without regard to age) in the United States experienced 10.37 fatal involvements per 161 million km (100 million mi) at night and 2.25 fatal involvements per 161 million km (100 million mi) during the day (Massie and Campbell, 1993). In their analysis, the difference between daytime and nighttime fatal rates was found to be more pronounced among younger age groups than among older ones, with drivers ages 20–24 showing a nighttime rate that was 6.1 times the daytime rate, and drivers age 75 and older showing a nighttime rate only 1.1 times the daytime rate. The lower percentage of nighttime crashes for older drivers may be due to a number of factors, including reduced exposure-older drivers as a group drive less at night—and a self-regulation process whereby those who do drive at night are the most fit and capable to perform all functional requirements of the driving task (National Highway Traffic Safety Administration, 1987).

A specific driving error with high potential for crash involvement is wrong-way movements. Analyses of wrong-way movements at intersections frequently associate these driving errors with low visibility and restricted sight distance (Vaswani, 1974, 1977; Scifres and Loutzenheiser, 1975) and note that designs that increase the visibility of access points to divided highways and treatments that improve drivers' understanding of proper movements at these locations have been found to reduce the potential for crashes.

Inadequate night visibility, where the vehicle's headlights are the driver's primary light source, was reported by Vaswani (1977) as a factor that makes it more difficult for drivers to determine the correct routing at intersections with divided highways. Similarly, Woods, Rowan, and Johnson (1970) reported that locations where highway structures, land use, natural growth, or poor lighting conditions reduce the principal sources of information concerning the geometry and pavement markings are associated with higher occurrences of wrong-way maneuvers. Crowley and Seguin (1986) reported that drivers over the age of 60 are excessively involved in wrong-way movements on a per mile basis. Suggested countermeasures include increased use of fixed lighting installations. Lighting provides a particular benefit to older drivers by increasing the expectancy of needed vehicle control actions at longer preview distances. It has been documented extensively in this Handbook that an older driver's ability to safely execute a *planned* action is not significantly worse than that of a younger driver. The importance of fixed lighting at intersections for older drivers can therefore be understood in terms of both the diminished visual capabilities of this group and their special need to prepare farther in advance for unusual or unexpected aspects of intersection operations or geometry. Targets that are especially critical in this regard include shifting lane alignments; changing lane assignments (e.g., when a through lane changes to turn-only operation); a pavement width transition, particularly a reduction across the intersection; and, of course, pedestrians.

Detectability of a pedestrian is generally influenced by contrast, motion, color, and size (Robertson, Berger, and Pain, 1977). If a pedestrian is walking at night and does not have good contrast, color contrast, or size relative to other road objects, an increase in contrast will significantly improve his/her detectability. This is one effect of street lighting. Extreme contrasts as well as dark spots are reduced, giving the driver and the pedestrian a more "even" visual field. The effectiveness of fixed lighting in improving the detectability of pedestrians has been reported by Pegrum (1972); Freedman, Janoff, Koth, and McCunney (1975); Polus and Katz (1978); and Zegeer (1991).

While age-related changes in glare susceptibility and contrast threshold are currently accounted for in lighting design criteria, there are other visual effects of aging that are currently excluded from visibility criteria. Solid documentation exists of age-related declines in ocular transmittance (the total amount of light reaching the retina), particularly for the shorter wavelengths (see Ruddock, 1965); this suggests a potential benefit to older drivers of the "yellow tint" of high-pressure sodium highway lighting installations. The older eye experiences exaggerated intraocular scatter of light—all light, independent of wavelength (Wooten and Geri, 1987)—making these drivers more susceptible to glare. Diminished capability for visual accommodation makes it harder for older observers to focus on objects at different distances. Pupil size is reduced among older individuals through senile miosis (Owsley, 1987), which is most detrimental at night because the reduction in light entering the eye compounds the problem of light lost via the ocular media, as described above.

The loss of static and dynamic acuity—the ability to detect fine detail in stationary and moving targets—with advancing age is widely understood. Although there are pronounced individual differences in the amount of age-related reduction in static visual acuity, Owsley (1987) indicated that a loss of about 70 percent in this capability by age 85 is normal. Among other things, declines in acuity can be used to predict the distance at which text of varying size can be read on highway signs (Kline and Fuchs, 1993) under a given set of viewing conditions.

There are a number of other aspects of vision and visual attention that relate to driving. In particular, saccadic fixation, useful field of view, detection of motion in depth, and detection of angular movement have been shown to be correlated with driving performance (see Bailey and Sheedy, 1988, for a review). As a group, however, these visual functions do not appear to have strong implications for highway lighting practice, with the possible exception of the useful field of view. It could be argued that it would be advantageous to provide wider angle lighting coverage to increase the total field of view of older drivers. High-mast lighting systems can increase the field of view from 30 degrees to about 105 degrees (Hans, 1993). Such wide angles of coverage might have advantages for older drivers in terms of peripheral object detection. However, because high-mast lighting systems tend to sacrifice target contrast for increased field of view, opinion is divided about their application at intersections. Currently, field of view is not considered as a parameter that needs to be optimized in lighting system design for intersection applications.

Rockwell, Hungerford, and Balasubramanian (1976) studied the performance of drivers approaching four intersection treatments, differentiated in terms of special reflectorized delineators and signs versus illumination. A significant finding from observing 168 test approaches was that the use of roadway lighting significantly improved driving performance and earlier detection of the intersection, compared with the other treatments (e.g., signing, delineation, and new pavement markings), which showed smaller improvements in performance.

Finally, it must be emphasized that the effectiveness of intersection lighting depends upon a continuing program of monitoring and maintenance by the local authority. Guidelines published by AASHTO (1984) identify depreciation due to dirt on the luminaires and reduced lumen output from the in-service aging of lamps as factors that combine to decrease lighting system performance below design values. Maintained values in the range of 60 to 80 percent of initial design values are cited as common practice in this publication. With a particular focus on the needs of older drivers for increased illumination relative to younger motorists to accommodate the age-related sensory deficits documented earlier in this discussion, a recommendation logically follows that lighting systems be maintained to provide service at the 80-percent level—i.e., the upper end of the practical range—with respect to their initial design values.

P. Design Element: Pedestrian Crossing Design, Operations, and Control

Table 25. Cross-references of related entries for pedestrian crossing design, operations, and control.

Applications in Standard Reference Manuals				
***MUTCD* (2000)**	**AASHTO *Green Book* (1994)**	***Roadway Lighting Handbook* (1978)**	**NCHRP 279, *Intersection Channelization Design Guide* (1985)**	***Traffic Eng. Handbook.* (1999)**
Sect. 1A.13, *crosswalk, crosswalk lines, & pedestrian* Tables 2A-1 & 2C-2 through 2C-3 Sects. 1A.14, 2B.39, 2B.40, 2C.37, 3B.15 & 3B.17 Figs. 3B-15 & 3B-16 Sect. 3B.19 Sect. 3E.01 Sect. 4A.02 Sects. 4C.05 & 4D.03 Sect. 4D.16 Sects. 4E.01 through 4E.09 Fig. 4E-1 Sect. 7A.03 Fig. 7A-1 Table 7B-1 Sects. 7B.08 & 7B.09 Fig. 7B-1 Sects. 7C.03, 7E.01 through 7E.09, 10C.01 & 10D.08	pp. 97-104, Sects. on *General Considerations, General Characteristics, Physical Characteristics, Walkway Capacities, Intersections, Characteristics of Persons With Disabilities, & Conclusions* p. 350, Para. 5 pp. 390-403, Sects. on *Separations for Pedestrian Crossings & Curb-Cut Ramps* p. 437, Sect. on *Curb-Cut Ramps* pp. 531-532, Sect. on *Pedestrian Facilities & Curb-Cut Ramps* p. 664, Para. 4 pp. 668-672, Sect. on *Effect of Curb Radii on Turning Paths* p. 679, Sect. on *Refuge Islands* p. 792, Sect. on *Wheel-Chair Ramps at Intersections*	p. 2, 2nd col., Para. 1 p. 9, Sect. on *Contrast* p. 18, Form 2 pp. 21-22, Tables 1-2 pp. 27-30, Sect. on *Warrants for Application of Specialized Crosswalk Illumination* p. 45, Para. 3 p. 71, 3rd bullet pp. 94-99 Sects. on *Coordination of the Arterial Lighting System and Traffic Controls & Pedestrian Lighting on Arterials*	p. 1, Item 3, 2nd bullet p. 6, Table 2-2 p. 9, Fig. 2-4 p. 11, Bottom left photo p. 21, Item 9 & Fig. 3-1 p. 27, Bottom left fig. p. 33, Bottom right fig. p. 38, Para. 1 & top two figs. p. 39, Entire page p. 40, Fig. 4-1 p. 42, 2nd col., item 3 p. 61, 2nd col, Paras. 2-3 & item 2 p. 66, Paras. 1 & 4 & item 4 p. 67, 2nd col, Paras. 1-2 p. 68, Fig. 4-26 p. 69, 2nd col., Para. 4 pp. 70-72, Figs. 4-27 & 4-28 p. 75, Paras. 4-5, Table 4-10, & 2nd col, Para. 1 p. 76, Item 4 & Table 4-11 pp. 96-97, Intersect. No. 4 pp. 104-105, Intersect. No. 9 pp. 150-151, Intersect. No. 36	pp. 41-42, Sect. on *Walking Speed* p. 43, Sect. on *Handicapped Pedestrians* pp. 46-47, Sect. on *Older Pedestrians* p. 385, Item 9 p. 409, Sect. on *Intersection Treatments* p. 414, Item 5 pp. 434-435, Sect. on *Crosswalks* pp. 497-498, Sect. on *Pedestrian Signal Heads* pp. 519-520, Sect. on *Pedestrian Detectors*

A nationwide review of fatalities during 1985 and injuries during 1983–1985 showed that 39 percent of all pedestrian fatalities and 9 percent of all pedestrian injuries involved persons age 64 and older (Hauer, 1988). While the number of injuries is close to the population distribution (approximately 12 percent), the number of fatalities far exceeds the proportion of older pedestrians. The percentages of pedestrian fatalities and injuries occurring at intersections were 33 percent and 51 percent, respectively (Hauer, 1988). People age 80 and older have the highest pedestrian death rate per 100,000 people; furthermore, the 1998 pedestrian death rate among men age 80 and older was more than three times as high as that for men age 74 and younger (IIHS, 2000). Crash types that predominantly involve older pedestrians at intersections are as follows (Blomberg and Edwards, 1990):

- Vehicle turn/merge—The vehicle turns left or right and strikes the pedestrian.
- Intersection dash—A pedestrian appears suddenly in the street in front of an oncoming vehicle at an intersection.

- Multiple threat-One or more vehicles stop in the through lane, usually at a crosswalk at an unsignalized intersection. The pedestrian steps in front of the stopped vehicle(s) and into the path of a through vehicle in the adjacent lane.
- Bus stop-related-The pedestrian steps out from in front of a stopped bus and is struck by a vehicle moving in the same direction as the bus.
- Pedestrian trapped-At a signalized intersection, a pedestrian is hit when a traffic signal turns red (for the pedestrian) and cross-traffic vehicles start moving.
- Nighttime-A pedestrian is struck at night when crossing at an intersection.

Earlier analyses of more than 5,300 pedestrian crashes occurring at urban intersections indicated that a significantly greater proportion of pedestrians age 65 and older were hit at signalized intersections than any other group (Robertson, Berger, and Pain, 1977).

Age-related diminished capabilities, which may make it more difficult for older pedestrians to negotiate intersections, include decreased contrast sensitivity and visual acuity, reduced peripheral vision and useful field of view, decreased ability to judge safe gaps, slowed walking speed, and physical limitations resulting from arthritis and other health problems. Older pedestrian problem behaviors include a greater likelihood to delay before crossing, to spend more time at the curb, to take longer to cross the road, and to make more head movements before and during crossing (Wilson and Grayson, 1980).

Older and Grayson (1972) reported that although older pedestrians involved in crashes looked more often than the middle-aged group studied, more than 70 percent of the adults struck by a vehicle reported not seeing it before impact. Job, Haynes, Prabhaker, Lee, and Quach (1992) found that pedestrians over age 65 looked less often *during* their crossings than did younger pedestrians. In a survey of older pedestrians (average age of 75) involved in crashes, 63 percent reported that they failed to see the vehicle that hit them or failed to see it in time to take evasive action (Sheppard and Pattinson, 1986). Knoblauch, Nitzburg, Dewar, Templer, and Pietrucha (1995) noted that difficulty seeing a vehicle against a (complex) street background may occur with vehicles of certain colors, causing them to blend in with their background. This is especially problematic for older persons with reduced contrast sensitivity, who require a higher contrast for detection of the same targets than younger individuals and who also have greater difficulty dividing attention between multiple sources and selectively attending to the most relevant targets. In addition, the loss of peripheral vision increases an older pedestrian's chances of not detecting approaching and turning vehicles from the side.

Reductions in visual acuity make it more difficult for older pedestrians to read the crossing signal. In a survey of older pedestrians in the Orlando, Florida area, 25 percent of the participants reported difficulty seeing the crosswalk signal from the opposite side of the street (Bailey, Jones, Stout, Bailey, Kass, and Morgan, 1992). Older pedestrians wait for longer gaps between vehicles before attempting to cross the road. In one study, approximately 85 percent of the pedestrians age 60 and older required a minimum gap of 9 s before crossing the road, while only 63 percent of all pedestrians required this minimum gap duration (Tobey, Shungman, and Knoblauch, 1983). The decline in depth perception may contribute to older persons' reduced ability to judge gaps in oncoming traffic. It may be concluded from these studies that older pedestrians do not process information (presence, speed, and distance of other vehicles) as efficiently as younger pedestrians, and therefore require more time to reach a decision. Other researchers have observed that older pedestrians do not plan their traffic behavior, are too trusting about traffic rules, fail to check for oncoming traffic before crossing at intersections, underestimate the speed of approaching vehicles, and follow other pedestrians without first checking for conflicts before crossing (Jonah and Engel, 1983; Mathey, 1983).

With increasing age, there is a concurrent loss of physical strength, joint flexibility, agility, balance, coordination and motor skills, and stamina. These losses contribute to slower walking speeds and difficulty negotiating curbs. In addition, older persons often fall as a result of undetected surface irregularities in the pavement and misestimation of curb heights. This results from a decline in contrast sensitivity and depth perception. In an assessment of 81 older residents (ages 70–97) to examine susceptibility to falling, 58 percent experienced a fall in the year following clinical assessment (Clark, Lord, and Webster, 1993). Impaired cognition, abnormal reaction to any push or pressure, history of palpitations, and abnormal stepping were each associated with falling. Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin (1995) reported that locating the curb accurately and placing the foot is a matter of some care, particularly for the elderly, the very young, and those with physical disabilities.

The studies discussed below define the types of crashes in which older pedestrians are most likely to be involved, and under what conditions the crashes most frequently occur. In addition, the specific geometric characteristics, traffic control devices (including signs, signals, and markings), and pedestrian signals that seem to contribute to older pedestrians' difficulties at intersections are discussed. Zegeer and Zegeer (1988) stressed the importance of "tailoring" the most appropriate traffic control measures to suit the conditions at a given site. The effect of any traffic control measure is highly dependent on specific locational characteristics, such as traffic conditions (e.g., volumes, speeds, turning movements), pedestrian volumes and pedestrian mix (e.g., young children, college students, older adults, persons with physical disabilities), street width, existing traffic controls, area type (e.g., rural, urban, suburban), site distance, crash patterns, presence of enforcement, and numerous other factors.

Harrell (1990) used distance stood from the curb as a measure of pedestrian risk for intersection crossing. Observations of 696 pedestrians divided among 3 age groups (age 30 and under, ages 31–50, and age 51 and older) showed that the oldest group stood the farthest from the curb, that they stood even farther back under nighttime conditions, and that older females stood the farthest distance from the curb. The author used these data to dispel the findings in the literature that older pedestrians are not cognizant of the risks of exposure to injury from passing vehicles. Similarly, it may be argued that this behavior keeps them from detecting potential conflict vehicles and makes speed and distance judgments more difficult for them, while limiting their conspicuity to approaching drivers who might otherwise slow down if pedestrians were detected standing at the curbside at a crosswalk.

A study of pedestrian crashes conducted at 31 high pedestrian crash sections in Maryland between 1974 and 1976 showed that pedestrians age 60 and older were involved in 53 (9.6 percent) of the crashes, and children younger than age 12 showed the same proportions. The pedestrians age 60 and older accounted for 25.6 percent of the fatal crashes. Compliance with traffic control devices was found to be poor for all pedestrians at all study locations; it was also found that most pedestrians keyed on the moving vehicle rather than on the traffic and pedestrian control devices. Only when the traffic volumes were so high that it was impossible to cross did pedestrians rely on traffic control devices (Bush, 1986).

Garber and Srinivasan (1991) conducted a study of 2,550 crashes involving pedestrians that occurred in the rural and urban areas of Virginia to identify intersection geometric characteristics and intersection traffic control devices that were predominant in crashes involving older pedestrians. Crash frequency by location and age for the crashes within the cities showed that while the highest percentage of crashes involving pedestrians age 59 and younger occurred within 46 m (150 ft) from the intersection stop line, the highest percentage of crashes for pedestrians age 60 years and older (51.8 percent) occurred *within the intersection*.

More recently, Knoblauch, Nitzburg, Reinfurt, et al. (1995) reported that, compared with younger pedestrians, older adults are overinvolved in crashes while crossing streets at intersections. In their earlier analysis of the national Fatal Analysis Reporting System (FARS) data for the period 1980–1989, 32.2 and 35.3 percent of the deaths for pedestrians ages 65–74 and age 75 and older, respectively, occurred at intersections (Reinfurt, Council, Zegeer, and Popkin, 1992). This compared with 22 percent or less for the younger age groups. Analysis of the North Carolina motor vehicle crash file for 1980–1990 displayed somewhat smaller percentages, but showed the trend of increasing pedestrian crashes at intersections as age increased. Further analysis of the North Carolina database showed that pedestrians age 65 and older, as well as those ages 45–64, experienced 37 percent of their crashes on roadways with four or more lanes. This compares with 23.7 percent for pedestrians ages 10–44 and 13.6 percent for those age 9 and younger. The highest number of pedestrian-vehicle crashes occurred when the vehicle was going straight (59.7 percent), followed by a vehicle turning left (17.2 percent), and a vehicle turning right (13.3 percent). Right-turn crashes accounted for 18.9 percent of crashes with pedestrians ages 65–74, compared with 14.2 percent for pedestrians age 75 and older. The oldest pedestrian group was the most likely to be struck by a left-turning vehicle; they accounted for 23.9 percent of the crashes, compared with 18.1 percent for those ages 65–74 and 15.8 percent for those ages 45–64.

Knoblauch, Nitzburg, Reinfurt, et al. (1995) conducted a study to determine if pedestrian comprehension of and compliance with pedestrian signals could be improved by installing a placard that explained the three phases of pedestrian signals. They used findings from: (1) a focus group and workshop conducted in Baltimore, Maryland, with 13 participants ages 19–62 and (2) questionnaires administered to 225 individuals ages 19–80 and older at four Virginia Department of Motor Vehicles offices to determine the most effective message content and format for a pedestrian signal education placard. The newly developed placard was installed at six intersections in Virginia, Maryland, and New York. Observational studies of more than 4,300 pedestrians during 600 signal cycles found no change in pedestrian signal compliance. However, results from questionnaires administered to 92 subjects at Departments of Motor Vehicles in Virginia, Maryland, and New York indicated a significant increase in understanding of the phases of the pedestrian signal. The authors concluded that although pedestrian crossing behavior is more influenced by the presence or absence of traffic than the signal indication, the wording on the placard was based on quantitative procedures using a relatively large number of subjects and should be used where signal educational placards are installed. The wording of the educational placard recommended by Knoblauch, Nitzburg, Reinfurt, et al. (1995) is shown in Recommendation 3 of Design Element P. A modification for a two-stage crossing is shown in Recommendation 4.

Zegeer and Cynecki (1986) tested a LOOK FOR TURNING VEHICLES pavement marking in a crosswalk, as a low-cost countermeasure to remind pedestrians to be alert for turning vehicles, including right-turn-on-red (RTOR) vehicles. Results showed an overall reduction in conflicts and interactions for RTOR vehicles and also for the total number of turning vehicles. Even with an RTOR prohibition, approximately 20 percent of the motorists committed an RTOR violation when given the opportunity (Zegeer and Cynecki, 1986). Of those violations, about 23.4 percent resulted in conflicts with pedestrians or vehicles on the side street.

Zegeer, Opiela, and Cynecki (1982) conducted a crash analysis to determine whether pedestrian crashes are significantly affected by the presence of pedestrian signals and by different signal timing strategies. They found no significant differences in pedestrian crashes between intersections that had standard-timed (concurrent walk) pedestrian signals compared with intersections that had no pedestrian signals. Concurrent or standard timing provides for pedestrians to walk concurrently (parallel) with traffic flow on the WALK signal. Vehicles are generally permitted to turn right (or left) on a green light while pedestrians are crossing on the WALK interval. Other timing strategies include early release timing, late release timing, and exclusive timing. In early release timing—also termed a "leading pedestrian interval"—the pedestrian WALK indication is given before the parallel traffic is given a green light, allowing pedestrians to get a head start into the crosswalk before vehicles are permitted to turn. In late release timing, the pedestrians are held until a portion of the parallel traffic has turned. Exclusive timing is a countermeasure where traffic signals are used to stop motor vehicle traffic in all directions simultaneously for a phase each cycle, while pedestrians are allowed to cross the street. "Barnes Dance" or "scramble" timing is a type of exclusive timing where pedestrians may also cross diagonally in addition to crossing the street. Exclusive timing is intended to virtually eliminate turning traffic or other movements that conflict with pedestrians while they cross the street. In the Zegeer et al. (1982) analysis, exclusive-timed locations were associated with a 50 percent decrease in pedestrian crashes for intersections with moderate to high pedestrian volumes when compared with both standard-timed intersections and intersections that had no pedestrian signals. However, this timing strategy causes excessive delays to both motorists and pedestrians. Older road users (age 65 and older) recommended the following pedestrian-related countermeasures for pedestrian signs and signals, during focus group sessions held as part of the research conducted by Knoblauch, Nitzburg, Reinfurt, et al. (1995): (1) reevaluate the length of pedestrian walk signals due to increasingly wider highways, (2) implement more Barnes Dance signals at major intersections, and (3) provide more YIELD TO PEDESTRIANS signs in the vicinity of heavy pedestrian traffic.

Several studies have been conducted to determine whether regulatory signing aimed at turning motorists could reduce conflicts with pedestrians. Zegeer, Opiela, and Cynecki (1983) found that the regulatory sign YIELD TO PEDESTRIANS WHEN TURNING was effective in reducing conflicts between turning vehicles and pedestrians. They recommended that this sign be added to the *MUTCD* as an option for use at locations with a high number of pedestrian crashes involving turning vehicles. Zegeer and Cynecki (1986) found that the standard NO TURN ON RED sign with the supplementary WHEN PEDESTRIANS ARE PRESENT message was effective at several sites with low to moderate right-turning vehicle volumes. However, it was less effective when RTOR volumes were high. It was therefore recommended that the supplemental message WHEN PEDESTRIANS ARE PRESENT be added to the *MUTCD* as an accepted message that may be used with an NTOR sign when right-turn volume is light to moderate and pedestrian volumes are light or occur primarily during intermittent periods, such as in school zones. The supplemental message when added to the NTOR sign with the circular red symbol reduced total pedestrian conflicts at one site and increased RTOR usage (as desired, from 5.7 percent to 17.4 percent), compared with full-RTOR prohibitions. It was recommended that the supplemental message be added to the *MUTCD* for the NTOR sign with the circular red symbol, under low to moderate right-turning vehicle volumes and light or intermittent pedestrian volumes.

More recently, Abdulsattar, Tarawneh, and McCoy (1996) found that the TURNING TRAFFIC MUST YIELD TO PEDESTRIANS sign was effective in significantly reducing pedestrian-vehicle conflicts during right turns. The sign was installed at six marked crosswalks in Nebraska, where right-turning vehicle-pedestrian conflict data were collected before and after its installation in an observational field study. For the six study crosswalks combined, a conflict occurred in 51 percent of the observations in the before period, but in only 38 percent of the observations during the after period. The reductions in pedestrian-vehicle conflicts across the observation sites ranged from 15 to 30 percent, and were statistically significant.

Considering pedestrian walking times, section 4E.09 of the *MUTCD* (2000) indicates that a pedestrian clearance interval shall be provided immediately following the WALK indication, and should consist of a flashing DON'T WALK interval of sufficient duration to allow a pedestrian crossing in the crosswalk to leave the curb and travel at a normal walking speed of 1.2 m/s (4.0 ft/s) to at least the center of the farthest traveled lane, or to a median, before opposing vehicles receive a green indication. The *MUTCD* further states that, "where pedestrians who walk slower than normal or pedestrians who use wheelchairs routinely use the crosswalk, a walking speed of less than 1.2 m should be considered in determining the pedestrian clearance time."

Older pedestrian walking speed has been studied by numerous researchers. ITE (1999) reports walking speeds obtained by Perry (1992) for physically impaired pedestrians. Average walking speeds for pedestrians using a cane or crutch were 0.80 m/s (2.62 ft/s); for pedestrians using a walker, 0.63 m/s (2.07 ft/s); for pedestrians with hip arthritis, 0.68 to 1.11 m/s (2.24 to 3.66 ft/s); and for pedestrians with rheumatoid arthritis of the knee, 0.75 m/s (2.46 ft/s). Sleight (1972) determined that there would be safety justification for use of walking speeds between 0.91 and 0.99 m/s (3.0 to 3.25 ft/s), based on the results of a study by Sjostedt (1967). In this study, average adults and the elderly had walking speeds of 1.37 m/s (4.5 ft/s); however, 20 percent of the older pedestrians crossed at speeds slower than 1.2 m/s (4.0 ft/s). The 85th percentile older pedestrian walking speed in that study was 1.04 m/s (3.4 ft/s). A 1982 study by the Minnesota Department of Transportation found that the average walking speed of older pedestrians was 0.91

m/s (3.0 ft/s). In a study conducted in Florida, it was found that a walking speed of 0.76 m/s (2.5 ft/s) would accommodate 87 percent of the older pedestrians observed (ITE, undated). Weiner (1968) found an average rate for all individuals of 1.29 m/s (4.22 ft/s), and of 1.13 m/s (3.7 ft/s) for women only. A Swedish study by Dahlstedt (undated), using pedestrians age 70 and older, found that the 85th percentile comfortable crossing speed was 0.67 m/s (2.2 ft/s).

Interviews and assessments were conducted with 1,249 persons age 72 and older from the New Haven, CT community of Established Populations for Epidemiologic Studies of the Elderly to determine walking speeds and self-reported difficulty with crossing the street as pedestrians (Langlois, Keyl, Guralnik, Foley, Marottoli, and Wallace, 1997). The study population excludes persons in nursing homes or hospitals. In a telephone interview, 11.4 percent indicated that they had difficulty crossing the street. Reasons provided included insufficient time to cross and difficulty with right-turning vehicles. The mean walking speed for those reporting difficulty crossing the street was 0.38 m/s (1.25 ft/s); for those reporting no difficulty, the speed was 0.59 m/s (1.94 ft/s). Only 7.3 percent of the population had measured walking speeds ≥0.91 m/s (3 ft/s), and less than 1 percent had walking speeds of 1.2 m/s (4.0 ft/s).

Hoxie and Rubenstein (1994) measured the crossing times of older and younger pedestrians at a 21.85-m- (71.69-ft-) wide intersection in Los Angeles, CA, and found that older pedestrians (age 65 and older) took significantly longer than younger pedestrians to cross the street. In this study, the *average* walking speed of the older pedestrians was 0.86 m/s (2.8 ft/s), with a standard deviation of 0.17 m/s (0.56 ft/s); the average speed of the younger pedestrians was 1.27 m/s (4.2 ft/s), with a standard deviation of 0.17 m/s (0.56 ft/s). Of the 592 older pedestrians observed, 27 percent were unable to reach the curb before the light changed to allow cross-traffic to enter the intersection, and one-fourth of this group were stranded at least a full traffic lane away from safety. A study of crossing speeds by Coffin and Morrall (1995) limited to 15 pedestrians age 60 or older, at each of 6 crosswalk locations in Calgary, Canada, documented an 85th percentile walking speed of 1.0 m/s (3.28 ft/s) for midblock crosswalks and 1.2 m/s (4.0 ft/s) for crosswalks at signalized intersections. The authors note that the walking speed of older pedestrians varies according to functional classification, gender, and intersection type, and that approximately 95 percent of the pedestrians in this study would be accommodated using a design walking speed of 0.8 m/s (2.62 ft/s).

Much more extensive observations of pedestrian crossing behavior were conducted at two crosswalk locations at two intersections in Sydney, Australia (a major six-lane divided street and a side street), where the design crossing speed was changed from 1.2 m/s to 0.9 m/s (4.0 ft/s to 3.0 ft/s) (Job, Haynes, Quach, Lee, and Prabhaker, 1994). Observations were made during 3,242 crossings during a baseline period (1.2-m/s [4.0-ft/s] design crossing speed) and 2 and 6 weeks after the flashing DON'T WALK interval was extended to allow for the slower crossing speed under study. This study was conducted to evaluate countermeasures to address the over-representation of pedestrians age 70 and older in crashes in the greater Sydney metropolitan area. At all crosswalk locations, the WALK phase remained a constant 6 s, and the clearance interval was extended from 14 s to 20 s at one intersection 18.2 m (59.7 ft) wide, and from 18 to 20 s at the other intersection measuring 24.2 m (79.4 ft) wide. Observations were conducted for 2,377 pedestrians ages 20-59, 511 pedestrians ages 60-65, and 354 pedestrians age 66 and older. The number of males and females was approximately equal. For both intersections, a general trend showed that the older the pedestrian, the longer the crossing time. Also, females crossed more slowly than males in all age groups. At the wider intersection, mean crossing speeds were 1.5 m/s (4.9 ft/s) for pedestrians ages 20-59; 1.3 m/s (4.27 ft/s) for pedestrians ages 60-65, and 1.1 m/s (3.6 ft/s) for pedestrians age 66 and older. The mean walking speed for females age 66 and older was 1.0 m/s (3.28 ft/s). The authors note that the assumed walking speed of 1.2 m/s (4.0 ft/s) leaves almost 15 percent of the total population walking below the assumed speed. Extending the clearance interval resulted in a decrease in the percentage of pedestrian-vehicle conflicts, from 4 percent in the baseline period to 1 percent in the experimental period at 2 weeks and also 1 percent at 6 weeks, at the wider intersection. This difference was significant at the $p \leq .001$ level. Observed changes in pedestrian-vehicle conflicts at the smaller intersection were contaminated by an increase in the proportion of pedestrians (in the young and young/middle age groups only) who crossed illegally (began to cross during the flashing DON'T WALK phase); consequently, sustained differences between the baseline and experimental phases were not demonstrated. At the conclusion of this research, the authors recommended a reduction in the design walking speed from 1.2 m/s to 0.9 m/s (4.0 ft/s to 3.0 ft/s) at locations where there is significant usage by older pedestrians.

Knoblauch, Nitzburg, Dewar, et al. (1995) conducted a series of field studies to quantify the walking speed, start-up time, and stride length of pedestrians younger than age 65 and pedestrians 65 and older under varying environmental conditions. Analysis of the walking speeds of 3,458 pedestrians younger than age 65 and 3,665 pedestrians age 65 and older crossing at intersections showed that the mean walking speed for younger pedestrians was 1.51 m/s (4.95 ft/s) and for older pedestrians, the mean walking speed was 1.25 m/s (4.11 ft/s). The 15th percentile speeds were 1.25 m/s and 0.97 m/s (4.09 ft/s and 3.19 ft/s) for younger and older pedestrians, respectively. These differences were statistically significant. Among the many additional findings with regard to walking speed were the following: (1) pedestrians who start on the WALK signal walk slower than those who cross on either the flashing DON'T WALK or steady DON'T WALK; (2) the slowest walking speeds were found on local streets, while the faster walking speeds were found on collector-distributors; (3) sites with symbolic pedestrian signals had slower speeds than sites with word messages; (4) pedestrians walk faster where RTOR is not permitted, where there is a median, and where there are curb cuts; and (5) faster crossing speeds were found at sites with moderate traffic volumes than at sites with low or high vehicle volumes.

For design purposes, a separate analysis was conducted by Knoblauch, Nitzburg, Dewar, et al. (1995) for pedestrians who complied with the signal, as they tended to walk more slowly than those who crossed illegally. The mean crossing speed for the young compliers was 1.46 m/s (4.79 ft/s) and for the older compliers, it was 1.20 m/s (3.94 ft/s). The 15th percentile speed for the young compliers was 1.21 m/s (3.97 ft/s) and was 0.94 m/s (3.08 ft/s) for the older compliers. Older female compliers showed the slowest walking speeds, with a mean speed of 1.14 m/s (3.74 ft/s) and a 15th percentile of 0.91 m/s (2.97 ft/s). One of the slowest 15th percentile values (0.89 m/s [2.94 ft/s]) was observed for older pedestrians crossing snow-covered roadways. It was concluded from this research that a mean design speed of 1.22 m/s (4.0 ft/s) is appropriate, and where a 15th percentile is appropriate, a walking speed of 0.91 m/s (3.0 ft/s) is reasonable. It was also determined by Knoblauch, Nitzburg, Dewar, et al. (1995) that the slower walking speed of older pedestrians is due largely to their shorter stride lengths. The stride lengths of all older pedestrians are approximately 86 percent of those of younger pedestrians.

Knoblauch, Nitzburg, Dewar, et al. (1995) also measured start-up times for younger and older pedestrians who stopped at the curb and waited for the signal to change before starting to cross. The mean value for younger pedestrians was 1.93 s, compared with 2.48 s for older pedestrians. The 85th percentile value of 3.06 s was obtained for younger pedestrians, compared with 3.76 s for older pedestrians. For design purposes, the authors concluded that a mean value of 2.5 s and an 85th percentile value of 3.75 s would be appropriate. These data specifically did not include pedestrians using a tripod cane, a walker, or two canes; people in wheelchairs; or people walking bikes or dogs. The *MUTCD* (2000) states that the WALK interval should be at least 7 s long so that pedestrians will have adequate opportunity to leave the curb or shoulder before the pedestrian clearance time begins (where pedestrian volumes and characteristics do not require 7 s, a 4-s interval may be used). Parsonson (1992) noted that the reason this much time is needed is because many pedestrians waiting at the curb watch the traffic, and not the signals. When they see conflicting traffic coming to a stop, they will then look at the signal to check that it has changed in their favor. If they are waiting at a right-hand curb, they will often take time to glance to their left rear to see if an entering vehicle is about to make a right turn across their path. Parsonson reported that a pedestrian reasonably close to the curb and alert to a normal degree can be observed to require up to 4 or 5 s for this reaction, timed from when the signal changes to indicate that it is safe to cross, to stepping off the curb. It may be remembered that older pedestrians stand farther away from the curb, and may or may not be alert. In addition, there are many drivers who run the amber and red signals, and it is prudent for pedestrians to double-check that traffic has indeed obeyed the traffic signal, and that there are no vehicles turning right on red or (permissive) left on green before proceeding into the crosswalk. Because older persons have difficulty dividing attention, this scanning and decision-making process requires more time than it would for a younger pedestrian. Parsonson (1992) reported that the State of Delaware has found that pedestrians do not react well to the short WALK and long flashing DON'T WALK timing pattern. They equate the flashing with a vehicle yellow period. The Florida Department of Transportation and the city of Durham, Ontario, provide sufficient WALK time for the pedestrian to reach the middle of the street, so that the pedestrian will not turn around when the flashing DON'T WALK begins.

One strategy that in recent implementations has appeared to offer promise in assisting pedestrians who are slower or more reluctant to cross when there is a perceived likelihood of conflict with turning vehicles is the leading pedestrian interval (LPI). An LPI is a brief, exclusive signal phase dedicated to pedestrian traffic. Van Houten, Retting, Farmer, and Van Houten (1997) investigated the effects of a 3-s LPI on pedestrian behavior and conflicts with turning vehicles at three urban intersections in St. Petersburg, FL. In the study, pedestrian-vehicle conflicts were observed during a baseline period, where the signal phasings at each intersection provided the onset of the pedestrian WALK signal and the onset of the green signal for turning vehicles concurrently. During the experimental phase, a 3-s LPI was installed to release pedestrian traffic 3 s before turning vehicles. The LPI was implemented using a modified, solid-state plug-in signal load switch that had the capacity to delay the change of the traffic signal phase from red to green. Pedestrians estimated to be age 65 and older were scored separately from those estimated to be age 12 and older. A total of 1,195 seniors and 3,680 non-seniors were observed across all three sites during the baseline condition. During the LPI condition, 860 seniors and 4,288 non-seniors were observed.

Observers collected data between 8:30 a.m. and 5:00 p.m., and scored the number of pedestrians who left the curb within 2 s before the start of the WALK indication, within 3 s after the onset of the WALK indication, during the remainder of the WALK cycle, and during the flashing DON'T WALK indication. The number of conflicts was scored for each of these intervals, defined as any situation where a driver engaged in abrupt braking or either the driver or pedestrian took sudden evasive action to avoid a collision. Conflicts were scored separately for right-tuning and left-turning vehicles. Other data of interest included the number of times that a pedestrian yielded to a turning vehicle by stopping or waving the vehicle through, and the distance covered by the pedestrian during the LPI condition. The intersection geometries included the following: (1) one-way traffic with four northbound lanes by two-way traffic with one lane in each direction and diagonal parking (north and west crosswalks were observed because both included left-on-green conflict potential); (2) one-way traffic with four southbound lanes by two-way traffic with one lane in each direction and diagonal parking (south and east crosswalks were observed because both included left-on-green conflict potential); and (3) two-way traffic with two lanes in each direction by two-way traffic with two lanes in each direction (all four crosswalks were observed).

The number of conflicts per 100 pedestrians who started crossing during a defined 5-s begin-walk period (which began 2 s before and ended 3 s after the onset of the WALK indication) showed that during the baseline period, the number of conflicts averaged 3.0, 2.1, and 3.3 for the three sites. After the introduction of the LPI, the number of conflicts averaged 0.1, 0.1, and 0.2 for the three sites. The likelihood of conflict was significantly lower during the LPI condition than during the baseline condition for both left- and right-turning vehicles; the odds of conflict for pedestrians leaving the curb during the begin-walk period were reduced by approximately 95 percent. The reduction in the odds of a conflict for seniors as a function of an LPI phase (89-percent reduction) was not significantly different from that of their younger counterparts (97-percent reduction). There was no significant effect of LPI on the odds of a conflict for pedestrians leaving the curb *after* the begin-walk period, indicating that an LPI does not move conflicts to a later phase in the WALK interval.

The LPI also had the effect of significantly reducing the number of pedestrians yielding to turning vehicles; the odds of a pedestrian yielding to a turning vehicle were reduced by approximately 60 percent. Van Houten et al. (1997) indicate that once pedestrians were in the crosswalk, drivers acknowledged their presence and were more likely to yield the right-of-way. Also, they state that pedestrians occupying the crosswalk were more visible to drivers who were waiting for the light to change than they would have been had the drivers and pedestrians been released concurrently. The final measure of interest was the mean distance traveled by the lead pedestrian during the LPI condition, which averaged 2.6 m (8.5 ft). The authors state that this distance (which is greater than one-half of a lane width) appears sufficient for pedestrians to assert their right-of-way ahead of turning vehicles, and reduces conflicts that may result when pedestrians and vehicles begin to move at the same time.

Q. Design Element: Roundabouts

Table 26. Cross-references of related entries for roundabouts.

Applications in Standard Reference Manuals		
MUTCD (2000)	*Highway Capacity Manual* (1998)	*Roundabouts: An Informational Guide* (2000)
Sects. 3B.16 & 3B.24 Figs. 3B-26 & 3B-27	p. 10-81, Items 3 & 7-10 p. 10-82, Fig. 10-37	pp. 1-5 & 1-6, Exhibits 1-3 & 1-4 (*Entry Width, Circulatory Roadway Width, & Inscribed Circle Diameter*) p. 1-7, Exhibit 1-5, Items c & d p. 1-11, Exhibit 1-6, Item d p. 1-13, Exhibit 1-7 p. 1-16, Sect. 1.6.4 p. 1-18, Sect. 1.6.6 p. 2-1, Exhibit 2-1 pp. 2-3 through 2-5, Sect. 2.1.1.1 p. 2-10, Para. 3 p. 2-13, Para. 6 p. 2-17, Para. 3 p. 3-1, Bullet *Steps 2 & 5* p. 3-3, Last two bullets on page. p. 3-5, Sects. 3.3 & 3.3.1 p. 3-11, Item 3 pp. 3-19 & 3-20, Para. 2 on 1st page. & Exhibits 3-14 & 3-16 pp. 4-1 & 4-2, Sect. 4.1.1 pp. 4-5 & 4-6, Paras. 3 & 5 on 1st page & Sect. 4.3.1 pp. 4-7 - 4-9, Sects. 4.3.4 & 4.3.5 p. 5-1, 1st bullet pp. 5-2 - 5-4, Sect. 5.2.11 p. 5-7, Para. 2 p. 5-8, Para. 1 p. 5-10, Exhibit 5-9 p. 5-15, Para. 1 p. 5-19, Para. 3 p. 5-20, *Entry-Circulating* Equation p. 5-21, 1st & 2nd bullets pp. 6-5 & 6-6, Sect. 6.2.1.3 pp. 6-17 through 6-21, Sects. 6.3.1, 6.3.2, & 6.33.1 p. 6-22, Para. 3 pp. 6-26 - 6-28, Sects. 6.3.7 & 6.3.8 pp. 6-47 & 6-48, Sects. 6.5.2 & 6.5.3 p. 6-51, Para. 2 Glossary: *Central Island, Circulatory Roadway Width, Entry Width, Inscribed Circle Diameter, Pedestrian Refuge, & Splitter Island*

Countermeasures that have been suggested to reduce the occurrence of older driver crashes at intersections have included changes to intersection operations (e.g., protected left-turn phases, elimination of RTOR, redundant signing, etc.) and geometric design (e.g., full positive offset of opposite left-turn lanes, increases in turning radius for right turns, etc.). One proposed solution to reduce not only the frequency but also the severity of crashes at intersections is the installation of a modern roundabout (Harkey, 1995; Jacquemart, 1998). This countermeasure, it has been suggested, addresses problems that older drivers experience in judging speeds and gaps, understanding operational rules at complex intersections, and maneuvering through turns. Specifically, the following advantages of roundabouts for older road users have been postulated:

- Reductions in the speed of vehicles entering the intersection/circle-this makes it easier to choose an acceptable gap to merge into, removes the need to accelerate quickly that occurs after a conventional right turn, and results in lower severity crashes with less serious injuries.
- The left turn is completely eliminated.
- The larger curb radius improves maneuverability.
- A simplified decision-making process results from one-way operation, yield-at-entry, and a reduced number of conflict points compared to a conventional intersection.

- A potential for improved pedestrian safety results from shorter crossing distances, fewer possibilities for conflicts with vehicles, and lower vehicle speeds-however, there are many unresolved issues surrounding the use of these facilities by (elderly and visually impaired) pedestrians at this time.

At the same time, there are significant human factors concerns about special driving task demands associated with the geometric and operational characteristics of roundabouts and their novelty in this country. First, the driver approaching a roundabout must comprehend the prescribed movements and, in particular, the yield-on-entry operation, as conveyed by upstream signing. For some years to come, these traffic control devices will be novel to motorists; and older persons are at a disadvantage in responding to novel, unexpected stimuli. Upon closer approach, the appropriate speed and heading changes to conform to the splitter island's controlling channelization must be performed; and where increased crash experience has been documented following roundabout installation, as discussed below, excessive entry speeds have been the prevalent contributing factor. Again, vehicle control for smooth entry may be more challenging for older than for younger drivers. At the point of entry, depending upon the deflection angle of the splitter island, there are critical seconds where confirmation that no conflict exists with a vehicle already in the roundabout requires a glance orientation that well exceeds 90 degrees. The increased difficulty for older drivers of visual search at skewed intersections has been underscored elsewhere in this Handbook (see p.69).

During negotiation of a roundabout, the ability to share attention between path guidance; gap (headway) maintenance; and visual detection, recognition, comprehension, and decision-making associated with exit location cues is a near-continuous requirement, even for single-lane facilities. With multiple lanes, the avoidance of conflicts with adjacent vehicles places an exaggerated demand on motorists' attention-sharing abilities; and of course, the increased traffic volumes and speeds associated with these higher capacity installations pose still greater demands. In the absence of controlled studies in the use of roundabouts by older drivers, it can only be stated qualitatively that the information-processing capacity will be exceeded sooner for older than for younger persons, and that accommodation by some seniors—probably by reducing their speed while in the roundabout—is likely. This will detract from the operational benefits roundabouts are designed to produce, and may impact safety as well.

A better understanding of the operational and safety issues surrounding the use of roundabouts by older drivers and pedestrians depends upon crash data analyses from the limited number of existing facilities and controlled and observational research in this area. This will require time, and more and more of these facilities are expected to come into operation in the immediate future. Thus, recommendations about when and why to use roundabouts to accommodate older road users remain premature, but an understanding of the roundabout task demands that pose special difficulty for seniors allows for certain recommendations regarding preferred practices when a jurisdiction has decided to install a roundabout. The recommendations presented for this design element attempt to balance the human factors considerations above with the accumulating body of information supporting roundabout usage (discussed below).

AASHTO does not maintain standards for the design of roundabouts; however, FHWA has recently developed a document entitled, *Roundabouts: An Informational Guide* (FHWA, 2000). The *Highway Capacity Manual* (1997) includes a proposed capacity formula for roundabouts. Presently, only several States have design guidelines for roundabouts (Florida, 1996 and Maryland, 1995) based largely on Australian guidelines. Both Florida and Maryland use SIDRA software (Australian methodology) to conduct an analysis of the capacity of a planned roundabout, which is available through McTrans at the University of Florida at Gainesville. A guide written for the California Department of Transportation Caltrans by Ourston and Doctors (1995) is based on British standards; according to Jacquemart (1998), Caltrans decided not to publish it. However, Caltrans has distributed a Design Information Bulletin (No. 80) to provide general guidance to project engineers on appropriate applications, site requirements, geometric elements, and traffic analysis. New York State is developing an *Engineering Instruction* (EI) on roundabouts that will base design guidance on British guides and software (RODEL). The EI notes that other software programs are permitted (e.g., Highway Capacity, SIDRA, ARCADY), provided that a RODEL analysis is performed for comparison purposes. This EI is to provide interim guidance for current projects, and will be incorporated into the NY State Highway Design Manual.

Flannery and Datta (1996) indicate that roundabouts are commonly used in Australia, Denmark, France, Germany, Great Britain, Ireland, The Netherlands, Norway, Portugal, South Africa, Spain, Sweden, and Switzerland. Sarkar, Burden, and Wallwork (1999) state that modern roundabouts are gaining in popularity in cities across the United States. (Arizona, California, Colorado, Florida, Kansas, Maryland, Massachusetts, Nevada, Oregon, Texas, and Wisconsin) because of their success in reducing speeds and the number of collisions. Because speeds are reduced, crashes are less severe. Because perpendicular left and right turns are eliminated, a roundabout with one-lane entries has fewer potential conflict points than a conventional intersection (8 vehicle-to-vehicle conflicts and 8 vehicle-to-pedestrian conflicts for a roundabout with 4 one-lane entries, compared to 32 vehicle-to-vehicle conflicts and 24 vehicle-to-pedestrian conflicts for a conventional four-leg intersection). Jacquemart (1998) reports that as of mid-1997, there were fewer than 50 modern roundabouts in the United States, compared to more than 35,000 in the rest of the world, with France owning the leading number of roundabouts (15,000 modern roundabouts currently, growing at a rate of 1,000 per year).

Flannery and Datta (1996) highlight the fact that modern roundabouts are different than earlier rotaries and traffic circles common in the early 1900s. First, the modern roundabout requires drivers who are entering the circle to yield to traffic already in the circle (known as "offside priority"). Early roundabout operations gave priority to drivers entering the circle ("nearside priority"), which caused circulating traffic to come to a complete stop, resulting in grid-lock. As a result of nearside priority, Flannery and Datta state that the operational performance of traffic circles declined rapidly with the increase in traffic beginning in the 1950s. Because traffic engineers believed that the problem was increased volume as opposed to nearside priority, traffic circles were generally abandoned in the United States. Studies conducted in The Netherlands; Victoria, Australia; and Western Australia have found significant reductions in crashes and casualty rates (from 60 to 90 percent fewer) at roundabouts converted from the old priority to the yield-on-entry priority.

Two other improvements in modern roundabout design are *deflection*, which helps to slow entering vehicles, resulting in safer merges with the circulating traffic stream, and *flared approaches*, which helps to increase capacity by increasing the number of lanes on the approach (Flannery and Datta, 1996). Jacquemart (1998) describes deflection as: "No tangential entries are permitted and no traffic stream gets a straight movement through the intersection. Entering traffic points toward the central island, which deflects vehicles to the right, thus causing low entry speeds." The splitter island is the geometric feature that physically separates entering traffic from exiting traffic, and defines the entry angle, which deflects and slows entering traffic. Looking at flared approaches from the viewpoint of accommodating older driver needs for simplicity, one-lane approaches are likely to be easier to negotiate. In the NCHRP *Synthesis of Roundabout Practice in the United States*, Jacquemart (1998) notes that the safety benefits of roundabouts (from studies in Australia and Europe) seem to be greatest for single-lane roundabouts under rural conditions. Generally, safety benefits are related to the reduced speed in the roundabouts, the simplification of conflict points, and the "increased responsibility caused by the slower motion and the need to concentrate and yield, as compared to driver behavior in signalized intersections" (Jacquemart, 1998).

As noted earlier, studies performed to date to evaluate the safety performance of roundabouts have not included driver age as a variable. Flannery and Datta (1996) conducted a safety analysis of six sites in Florida, Maryland, and Nevada that were converted from conventional intersections with traditional control (one-way stop, two-way stop, or signalized) to roundabouts. All six sites had one-lane entrances and only one lane of circulating traffic. Five roundabouts had a posted speed of 56 km/h (35 mi/h) and one had a posted speed of 72 km/h (45 mi/h). Four of the sites had four approaches and two sites had three approach legs. Crash data were collected for a period of 1 to 3 years before and after retrofitting the sites (depending on location). Results of the chi-square and normal approximation statistical tests indicated that crash frequencies were significantly reduced in the period after the sites were retrofitted as modern roundabouts. The sites were not stratified by average daily traffic (ADT) or previous type of traffic control since the sample size was small; therefore, particular crash reduction factors were not identified. However, quick inspection of the crash frequencies provided by site indicates that only the roundabout retrofitted from a signalized intersection showed an increase in crashes in the after period; the other five sites (one-way and two-way stop-controlled) showed decreases in crash frequency in the after period (in the range of 60 to 70 percent). Analyses could only be performed on crash frequencies by group (as opposed to site), because traffic volumes before and after were not characterized, and the six retrofitted roundabouts varied in ADT from 4,069 to 17,825 vehicles.

Rahman (1995) and Jacquemart (1998) provided before and after crash data for the roundabout established in Lisbon, MD in 1993. In the 6 years prior to the roundabout, there were 45 reported intersection crashes, with an average of 8 crashes per year. From 1993 to 1995 (after roundabout installation), there were only two reported crashes. Before the roundabout, the crashes were almost all angle crashes, and after the roundabout was installed, one of the crashes was a single-vehicle crash against a fixed object, and the other crash was a rear-end crash. Injury crashes decreased from 4.3 per year to 0.3 Total delays decreased by 45 percent, from 1.2 vehicle-hours to 0.34 vehicle-hours in the morning peak hour and from 1.09 vehicle-hours to 0.92

vehicle-hours in the afternoon peak hour. This roundabout has four approach legs; it was retrofitted from a two-way stop-controlled (flashing red beacon) intersection. The ADT was 8,500 vehicles (in March 1995). The inscribed diameter is 30.5 m (100 ft); there are one-lane entries measuring 5.5 m (18 ft); there is one lane of circulating traffic that is 5.5 m (18 ft) wide; and in 1995, the peak-hour total approach volume was 630 (Jacquemart, 1998). Rahman (1995) states that, "the performance of this first experimental roundabout in Maryland demonstrates the safety of roundabouts when properly designed."

Jacquemart (1998) examined the before and after crash data from 11 roundabouts in the United States. Results are described for large roundabouts with three-lane entries (one in Long Beach, CA and two in Vail, CO) and smaller roundabouts with one- or two-lane entries and inscribed circle diameters of 37 m (121 ft) or less (Santa Barbara, CA; Lisbon, Cearfoss, Lothian, and Leeds, MD; Tampa, FL; Montpelier, VT; and Hilton Head, SC). He states that the small- to moderate- size roundabouts showed significant reductions in total crashes (from an average annual crash frequency of 4.8 to 2.4, or 51 percent) and injury crashes (from an average annual crash frequency of 2.0 to 0.5, or 73 percent). There were no statistically significant differences in property-damage-only (PDO) crashes at the smaller roundabouts, although there was a reduction from 2.4 to 1.6 average annual crashes, or 32 percent. Although there was a trend toward crash reduction for the larger roundabouts, there were no statistically significant reductions in total crashes, injury crashes, or PDO crashes. Each roundabout experienced a reduction in injury crashes ranging from 20 to 100 percent. PDO crashes increased at a roundabout in Vail, CO, from 15 to 18 per year, and at Leeds, MD, from 1.5 to 5.3 per year. At the other nine roundabouts, however, PDO crashes decreased from six to one per year. Although PDO crashes at the Leeds, MD site showed an increase, injury crashes decreased from 2.2 to 0.0 per year. The PDO crashes at this site were all single-vehicle crashes that occurred because the vehicles entered the roundabout too fast. Jacquemart (1998) reports findings by Niederhauser, Collins, and Myers (1997) who showed that the average cost per crash decreased by 30 percent across the five conventional intersections in Maryland that were retrofitted to roundabouts, from $120,000 before the roundabout to $84,000 after the roundabout.

Niederhauser, Collins, and Myers (1997) reported the before and after average annual crash history for the five intersections in Maryland that were converted to roundabouts. All sites are single-lane approach and single-lane circulating roundabouts. Overall, the average crash rate was reduced from an average of 5.0 crashes per year to an average of 2.4 crashes per year, which is a reduction of greater than 50 percent. Data for each roundabout are reported in table 27.

Persuad, Retting, Garder, and Lord (2000) evaluated the change in crashes following conversion of 24 intersections in urban, suburban, and rural environments in 8 States (California, Colorado, Florida, Kansas, Maine, Maryland, South Carolina, and Vermont) from stop sign or signal control to modern roundabouts. The Bayes procedure was used to account for regression to the mean and to normalize differences in traffic volume between the before and after periods. The number of months of crash data available in the before period ranged from 21 to 66, and the number of months of crash data available in the after period ranged from 15 to 68. Across all sites and crash severities, crashes were reduced by 39 percent in the after-conversion period. A 76-percent reduction was estimated in the after period for injury crashes. For the 20 sites where injury data were available, there were 3 fatal crashes in the before period and none in the after period. There were 27 incapacitating injury crashes in the before period and 3 in the after period. Thus, the estimated reduction in fatal and incapacitating injury crashes is 89 percent.

Table 27. Before-and-after average annual crash history for the five intersections in Maryland that were converted to roundabouts.
Source: Niederhauser, Collins, and Myers (1997).

Site	Average Annual Crashes	
	Before	**After**
Lisbon	6.0	2.0
Cearfoss	2.7	0
Leeds	3.3	4.9
Lothian	7.7	5.1
Taneytown	5.3	0

Persuad et al. (2000) looked at the crash reduction rates as a function of operating environment and before-conversion control. For the nine urban single-lane roundabouts converted from stop control, a 61-percent reduction was estimated for all crash severities combined, and a reduction of 77 percent was estimated for injury crashes. For the five rural single-lane roundabouts converted from stop control, a 58-percent reduction was estimated for all crash severities combined, and a reduction of 82 percent was estimated for injury crashes. For the seven urban multi-lane roundabouts, a 15-percent reduction in crashes of all severities was estimated. Injury data were not available for four of these sites in the before-conversion period. For the three roundabouts converted from traffic signal control, all crashes were reduced by 32 percent, and injury crashes were reduced by 68 percent. The authors note that the smaller safety effects for the group of urban multi-lane roundabouts suggests that there may be differences in safety performance for single-lane designs compared to multi-lane designs. However, they caution that all seven of these roundabouts were located in one State (Colorado) where three of the four in Vail, CO, are part of a freeway interchange that also includes nearby intersections that were previously four-way stop-controlled. Finally, in this research, pedestrian and bicycle crash samples were too small to be meaningful; however, there were three reported pedestrian crashes during the before period and one with minimal injuries in the after period. Four bicyclists were injured in the before period and three during the after period.

Wallwork (1993) notes that crashes do occur at roundabouts and consist of rear-end or merge-type crashes. Both crash types are low speed and low impact, and result in few-if any—injuries. He states that with a roundabout, "no one can 'run the red" and cause a right-angle collision, nor can drivers make a mistake in selecting a gap in the approaching through traffic when making a left turn. The only decision an entering driver needs to make is whether or not the gap in the approaching/circulating traffic is large enough to enter safely." Lower speeds (less than 40 km/h [25 mi/h]) result in shorter braking distances and longer decision-making times. Even if a driver makes a mistake and chooses a gap that is too short, a collision is easier to avoid. Thus, the reduction in task difficulty, coupled with the low-speed environment, results in an overall reduction in the number of crashes and a reduction in the severity of the crashes that do occur, which should be especially beneficial to older persons.

The delays before and after eight intersections (seven of which were two-way or multi-way stop-controlled, and one was signalized) were converted to roundabouts were also described by Jacquemart (1998). The total delay (stopped delay plus move-up time in queue) for eight U.S. roundabouts before retrofit was 13.7 s for morning peak time and 14.5 s for afternoon peak time. This compares to 3.1 s for morning and 3.5 s for afternoon peak times after conversion to roundabouts. Thus, delays were reduced by 78 percent in morning peak periods and by 76 percent in afternoon peak periods after intersections were converted to roundabouts.

Jacquemart (1998) received information about the design of 38 roundabouts in the United States, and presented data for four major geometric features: (1) inscribed circle diameter; (2) circulatory roadway width; (3) central island; and (4) entry widths. The *inscribed circle diameter* is defined as the circle that can be inscribed within the outer curbline of the circulatory roadway. Twenty-eight of the 31 roundabouts for which data were provided on this element have an inscribed circle diameter in the range of 30 to 61 m (98 to 200 ft), with the majority of these (11) ranging from 30 to 32.9 m (98 to 108 ft). Regarding *circulatory roadway width*, 43 percent of the cases are 4.5 to 5.5 m wide (15 to 18 ft) wide; 21 percent are 6.0 to 7.0 m (20 to 23 ft) wide; 25 percent are 7.3 to 9.1 m (24 to 30 ft) wide; and 11 percent are 10.7 to 11.0 m (35 to 36 ft) wide. Thus, 36 percent are at least two lanes wide. The *central island* can be raised or flush, or it can be raised with a sloping curb or drivable apron surrounding it. The truck apron is generally included in the central island diameter. Jacquemart reports that approximately 66 percent of the roundabouts for which data were provided have central islands greater than 9 m (30 ft) in diameter. Regarding *entry widths*, 59 percent of the reported cases have single-lane entries, 30 percent have two-lane entries, and 11 percent have three-lane or more entry legs. Studies in other countries help to shed some light on the optimum design characteristics of modern roundabouts.

In the Jacquemart (1998) synthesis, a study by Brilon (1996) of 34 modern roundabouts in Germany concluded that 30 m (98 ft) seemed to be the ideal inscribed diameter for a single-lane roundabout. Brilon states that smaller diameters result in larger circulatory roadways that reduce the deflection. In addition, truck aprons with a rougher pavement are recommended so that the circulatory roadway remains 4 to 4.5 m (13 to 15 ft) wide. In a study of 83 roundabouts in France (Centre D'Etudes Techniques de L'Equipment de l'Ouest, 1986) in Jacquemart (1998), it is also concluded that roundabouts with smaller diameters have fewer crashes than larger roundabouts or those with oval circles. Their data indicate that the 13 roundabouts with inscribed diameters of <30 m had a crash frequency of 0.69 crashes per roundabout. This compares to 1.54 crashes per roundabout for the 11 roundabouts with inscribed diameters of 30 to 50 m; 1.58 crashes per roundabout for the 26 roundabouts with inscribed diameters of 50 to 70 m; 1.81 crashes per roundabout for the 16 roundabouts with inscribed diameters of 70 to 90 m; 3.80

crashes per roundabout for the 8 roundabouts with inscribed diameters of 90 m or greater; and 4.40 crashes per roundabout for the 9 oval roundabouts.

Splitter islands are another geometric feature of modern roundabouts. These are generally raised islands that are placed within a leg of a roundabout to separate entering and exiting traffic and to deflect entering traffic. They also serve as a safety zone for pedestrians. Only one of the 38 roundabouts has painted (marked) splitter islands. The study conducted in Germany (Brilon, 1996, in Jacquemart, 1998) concluded that splitter islands are important to the safety of pedestrians, and should be 1.6 to 2.5 m (5 to 8 ft) wide, with pedestrian crossings located 4 to 5 m (13 to 16 ft) back from the circulating roadway. A study conducted in Switzerland by Simon and Rutz, 1988 (in Jacquemart, 1998) also concluded that the distance between the pedestrian crossing and the inscribed circle should be 5 m (16 ft) since greater distances do not increase pedestrian safety. They recommended the use of splitter islands with safety zones for pedestrians for crossings with more than 300 vehicles per hour. Wallwork (1999) states that a feature of roundabouts that makes them safer for pedestrian than conventional intersections is that pedestrians walk *behind* the cars. He recommends moving the crosswalk back one car length from the yield line for each lane of entry (i.e., one car length for a one-lane entry, two car lengths for a two-lane entry, or three car lengths for a three-lane entry). Brilon (1996) recommended zebra-striped crossings only when there were more than 100 pedestrians crossing during the peak hour. Maryland (DOT/SHA, 1995) normally places pedestrian crossings 6 to 7.6 m (20 to 25 ft) from the yield line. Crosswalk striping is not used in order to avoid driver confusion of crosswalk limit lines with yield lines. Special consideration is given in providing priority crossings for pedestrians where pedestrian volumes are high, where there is a high proportion of younger or older pedestrians, or where pedestrians experience particular difficulty in crossing and are being delayed excessively. The agency believes that it is desirable to place these crossings at least 23 m (75 ft) downstream of the exit from the roundabout and possibly augment the crossing with a signal. This will reduce the possibility that vehicles delayed at the pedestrian crossing will queue back into the roundabout and gridlock the whole intersection.

In the survey conducted by Jacquemart (1998) detailing 38 U.S. roundabouts, 56 percent of the sites were reported to have no or very few pedestrians, 22 percent have between 20 and 60 pedestrians during the peak hour, and 22 percent have more than 60 pedestrians per hour. Of particular interest is the Montpelier, VT roundabout, which is located next to a senior housing project and is also close to a middle school (400 students), and carries in excess of 260 pedestrians during each rush-hour (morning and afternoon) period on school days (Gamble, 1996; Redington, 1997). This roundabout has three legs, an inscribed diameter of 34 m, one-lane entries for each lane, and one lane of circulating traffic. The annual average daily traffic (AADT) is approximately 11,000 (7,300 for each leg) and carries approximately 40 tractor trailers (WB-62) each day (Redington, 1997). The peak-hour total approach volume is 1,000 vehicles (Jacquemart, 1998). Prior traffic control was a one-way stop at a Yintersection.

Jacquemart (1998) lists criteria to assist visually impaired pedestrians, including: (1) keeping the crossing away from the circle (e.g., 5 to 6 m from the outer circle) lets the blind person distinguish the exiting traffic from the circulating traffic; and (2) the splitter island provides a refuge where the pedestrian can shift his or her attention from one traffic stream to another. Different pavement texture for the walkways will assist the visually impaired pedestrian in locating the crosswalks. Drivers approaching a roundabout approach at speeds slower than they would for an approach to a conventional intersection; thus, they are more likely to stop for pedestrians, and may be more likely to notice a pedestrian on an approach to a roundabout because they are not concentrating on finding a gap in the opposing traffic stream in which to turn left.

Jacquemart (1998) also provided a summary of the current lighting, signing, and pavement marking practices at the 38 U.S. roundabouts for which questionnaire data were provided. First, all existing roundabouts were reported to have nighttime lighting. Next, all roundabouts were reported to have the standard YIELD sign, although it was often supplemented by an additional plate with specific instructions, such as TO TRAFFIC ON LEFT; TO TRAFFIC IN ROUNDABOUT; or TO TRAFFIC IN CIRCLE; or with the international roundabout symbol, which is three arrows in a circular pattern. In addition, 90 percent of the roundabouts contain an advance YIELD AHEAD symbol sign and 7 percent use the YIELD AHEAD legend sign. Twenty-four percent included a supplemental plate on the advance YIELD sign that said AT ROUNDABOUT, presented the roundabout symbol, or displayed a speed limit sign. All roundabouts had either a one-way sign (R6-1 or R6-2) or a large arrow warning sign (W1-6) in the central island. Chevron signs often accompanied the one-way signs (see figure 15). Regarding pavement markings, approximately 20 percent of the roundabouts supplemented the yield line at the roundabout entrance with the pavement marking legend YIELD or YIELD AHEAD. For multi-lane roundabouts, only in the case of the Hilton Head, SC, roundabout were lane lines present. Jacquemart (1998) reported that the authorities responsible for the roundabout believe that the large number of senior drivers in the area would be more comfortable with lane markings in the circle. Simon and Rutz, 1988 (in Jacquemart, 1998) recommended that for main roads or national highways, advance directional signs with the roundabout symbol should supplement the roundabout yield sign at the entry; however, other special warning signs-such as ROUNDABOUT AHEAD or PRIORITY TO THE LEFT-are *not* recommended. Wallwork (1999) does not recommend the widespread use of supplemental signs (e.g., posting TO TRAFFIC IN CROSSWALK on the YIELD sign), because it constitutes visual clutter. Instead, he recommends their use only as a local measure to educate road users for a short time period after roundabout installation.

Figure 15. One Way and chevron sign combination for use in central island of roundabout.

Maryland's practice (Maryland DOT/SHA,1995) for State highway and county collector roads is to provide the following signs on the approach:

- Junction assembly.
- ROUNDABOUT AHEAD warning signs, with YIELD AHEAD plates (see figure 16).

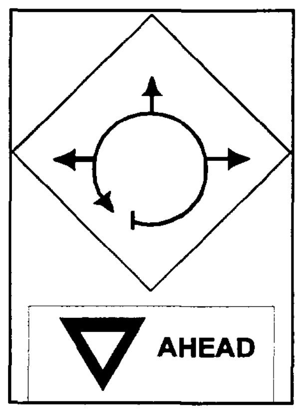

Figure 16. Signs used on approaches to Maryland roundabouts.

- Destination guide signs (either conventional verbal signs with arrows or, for higher speed multi-lane approaches, the use of diagrammatic guide signs).
- YIELD AHEAD signs (W3-2A) in combination with advisory speed plates (W13-1).
- Other guide signs, such as the Advance Route Marker Turn Assemblies.

At the roundabout intersection, the following signs are used:

- YIELD (R1-2) signs in combination with TO TRAFFIC IN CIRCLE (see figure 17).
  ONE WAY (R6-1R) signs in combination with obstruction markers (see figure 15).
- Exit guide signs.

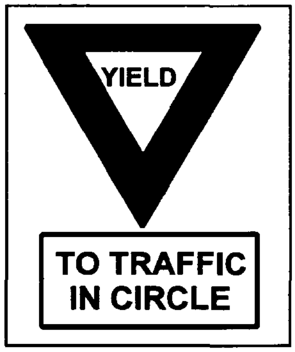

Figure 17. Signs used at Maryland roundabouts.

For local roadways, the following signs are recommended:

- ROUNDABOUT AHEAD warning signs, with YIELD AHEAD plates.
- Destination guide signs.
- YIELD (R1-2) signs in combination with TO TRAFFIC IN CIRCLE.
- ONE WAY (R6-1R) signs in combination with obstruction markers.
- Exit guide signs with DO NOT ENTER (R5-1) mounted on the back.

Maryland's pavement markings at roundabouts consist of:

- A 200-mm- to 400-mm- (8-in- to 16-in-) wide yield line, with 0.91-m (3-ft) segments and 0.91-m (3-ft) gaps that mark the entrance to the roundabout.
- 400-mm- (16-in-) wide solid yellow hatch markings in the splitter island envelope.
- Raised retroreflective pavement markers delineating the splitter island envelope.
- A 200-mm yellow solid pavement marking delineating the inner circle.
- A 200-mm- (8-in-) wide white edgeline delineating the right side of the roadway from the beginning of the splitter island to the yield line and a 200-mm- (8-in-) wide yellow pavement marking delineating the splitter island envelope.
- Optional rumble strips to reduce approach speeds, usually for high-speed rural approaches.

The decision to use lane lines in the circulating roadway for multi-lane roundabouts is made on a case-by-case basis, since it is believed by the agency that pavement markings may confuse rather than assist drivers in negotiating the roundabout.

Jacquemart lists several locations type where it is appropriate to install roundabouts, based on a review of guidelines from abroad and those existing guidelines in the United States. (e.g., Maryland and Florida). These locations include:

- High crash locations, particularly with high crash rates related to cross- movements or left-turn or right-turn movements.

- Locations with high delays.
- Four-way stop intersections.
- Intersections with more than four legs.
- Intersections with unusual geometry (Y or acute angle).
- Intersections with high left-turn flows.
- Intersections with changing traffic patterns.
- Intersections where U-turns are frequent or desirable along commercial corridors.
- At locations where storage capacities for signalized intersections are restricted or where the queues created by signalized intersections cause operational or safety problems.
- Intersections where the character or speed of the road changes, such as at entry points to a community or at junctions where a bypass road connects to an arterial.

Ourston and Bared (1995) cited the work of Guichet (1992) who investigated 202 crashes at 179 urban roundabouts in France. The crash causes and relative frequencies are presented in table 28.

Table 28. Causes of crashes at urban roundabouts in France.
Source: Ourston and Bared (1995).

Cause of Crash	**Percentage of Crashes**
Entering traffic fails to yield to circulating traffic	36.6
Loss of control inside the circulatory roadway	16.3
Loss of control at entries	10.0
Rear-end crashes at entries	7.4
Sideswipe, mostly at two-lane exits with bicyclists (2 out of 3 instances)	5.9
Running over pedestrians at marked crosswalks, mostly at two-lane entries	5.9
Pedestrians on the circulatory roadway	3.5
Loss of control at exits	2.5
Head-on collision at exits	2.5
Weaving inside the circulatory roadway	2.5

Guichet (1992) listed the major design recommendations, based on the findings of the crash investigation:

- Ensure that motorists recognize the approach to the roundabout.
- Avoid entries and exits with two or more lanes, except for capacity requirements.
- Separate the exit and entry by a splitter island.
- Avoid perpendicular entries or very large radii.
- Avoid very tight exit radii.

- Avoid oval-shaped roundabouts.

Wallwork (1999) recommends that in areas where there is a high concentration of senior drivers, it is desirable to use the lower end of the speed range that he has determined for roundabouts in a particular roadway class. He states that a roundabout meets drivers' requirements for simple decision-making, and a low speed is paramount for safe roundabout operation. His design-speed recommendations by roadway class are presented in table 29.

Table 29. Design-speed recommendations by roadway class for modern roundabouts. Source: Wallwork (1999).

Roadway Classification	Roundabout Design Speed
Local Road	19-24 km/h (12-15 mi/h)
Collector Road	24-29 km/h (15-18 mi/h)
Secondary Arterial	29-34 km/h (18-21 mi/h)
Major Arterial	34-37 km/h (21-23 mi/h)
Rural Roadway	Maximum 40 km/h (25 mi/h)

Wallwork states that the best way to control driver behavior is through the use of concrete: the roundabout has a concrete circle in the center, which defines a path to control speed, and a roundabout uses concrete islands to deter wrong-way movements and to control entry speeds. Roundabouts that are not designed for slow speeds result in high crash rates; there are at least two in the United States. (Boulder, CO and Daytona Beach, FL) that are being removed because of poor design (e.g., no bulbouts for deflection on the entries, allowing for 64-km/h [40-mi/h] speeds). One other feature of roundabouts that is important for all drivers, but for older drivers in particular, is high visibility. Wallwork recommends that tall trees, fountains, or statues be placed in the center of the roundabout so that long-range vision (at least 152 m [500 ft] of preview distance) of the roundabout is available. This will let a driver know that a reduction in speed is necessary downstream.

Regarding public opinion about roundabout implementation, Taekratok (1998) indicates that people do not make a clear distinction between modern roundabouts and traffic circles, and therefore public responses to roundabout proposals are negative. Jacquemart (1998) presents copies of media coverage (*Howard County Sun* newspaper) about the Lisbon, MD roundabout installed in Howard County as an experimental solution to an intersection with a high crash rate. One year before the roundabout opened, most of the Lisbon residents objected to the idea of a roundabout. Four months after the roundabout opened, a local citizen's committee voted overwhelmingly to make the roundabout permanent. Taekratok (1998) reports that the strategies used by Florida, Maryland, and Vermont have been successful in improving public perception; these include public education through the use of brochures, videotapes, and mass media to provide information during the development stage. This will help the public to understand the differences between traffic circles and roundabouts, and will gradually reduce opposition.

Redington (1997) notes that roundabouts are small (e.g., 28 to 55 m [91.8 to 180 ft]) compared to the old-time traffic circles found in New England and New Jersey (e.g., 76 m [249 ft] or greater), and that drivers strongly dislike traffic circles with their typical operating speeds of 50 to 60 km/h (31 to 41 mi/h). While the Montpelier, VT, Keck Circle Roundabout was under construction, the Roundabout Demonstration Committee prepared educational materials that included a brochure providing safety rules for drivers and pedestrians, as well as news releases and public service announcements in response to negative public reaction during construction and negative commentary from local morning radio personalities (Redington, 1997). This committee also conducted a survey of 111 citizens working or living near the roundabout 1 year after its opening to measure public opinion. Of the 111 respondents, 104 had driven the roundabout, 89 had walked, and 19 had bicycled. "Very favorable" or "favorable" responses were obtained from 57.6 percent of the respondents, 27.9 percent of the responses were "neutral,"and 14.4 percent were "unfavorable" or "very unfavorable." The survey contained two open-ended questions to allow respondents to contribute their likes, dislikes, and comments about "what they miss about the old intersection." The 111 respondents contributed 214 comments. The majority of the 65 "like" comments pertained specifically to smoother and better traffic movement. Fifty-six comments were obtained from respondents who dislike the roundabout. The majority of these were directed toward poor driver behavior, such as drivers failing to yield, failing to follow the rules, and failure to use turn signals.

Finally, Sarkar, Burden, and Wallwork (in press) reviewed driver's manuals for 32 States and the District of Columbia and concluded that the information on traffic circle and roundabout use was inadequate. Only 10 of the States provided some instruction in their manuals about how to use the traffic circles (i.e., entering drivers should yield to drivers who are already in the circle) and none provided information about how to use roundabouts. Information about types of signs placed near roundabouts and traffic circles was not present, nor was there any explanation about the differences between circles and roundabouts. Only one State had an illustration of a traffic circle, but in the authors' opinion, it was not clear or easy to understand. They recommend that State driver's manuals be revised to include information about the correct use of traffic circles and roundabouts, since roundabouts are becoming increasingly popular in the United States.

II. INTERCHANGES (GRADE SEPARATION)

The following discussion presents the rationale and supporting evidence for Handbook recommendations pertaining to these four design elements (A–D):

A. Exit Signing and Exit Ramp Gore Delineation
B. Acceleration/Deceleration Lane Design Features
C. Fixed Lighting Installations
D. Traffic Control Devices for Prohibited Movements on Freeway Ramps

A. Design Element: Exit Signing and Exit Ramp Gore Delineation

Table 30. Cross-references of related entries for exit signing and exit ramp gore delineation.

Applications in Standard Reference Manuals			
MUTCD (2000)		AASHTO *Green Book* (1994)	*Traffic Engineering Handbook* (1999)
Sects. 2A.12, 2A.14, 2A.17, 2B.03, 2B.29 through 2B.31, & 2B.46 Tables 2B-11 & 2C-4 Sects. 2C.33, 2D.05, 2D.06, 2E.09, 2E.10, 2E.11, & 2E.13 Tables 2E-1through 2E-4 Sects. 2E.18 through 2E.20 Figs. 2E-3 through 2E-7 Sect. 2E.20 Figs. 2E-8 through 2E-10 Sect. 2E.25	Fig. 2E-10 Sects. 2E.28 & 2E.30 through 2E.34 Figs. 2E-13 through 2E-19 Sects. 2E-40 through 2E-48 Figs. 2E-26 through 2E-30 Sect. 2E.50 Sect. 3B.05 Figs. 3B-8 & 3B-22 Sects. 3C.01, 3C.03, & 3D.01 through 3D.04 Figs. 6H-42 & 6H-43 plus accompanying notes	P. 276, Item 8 P. 897, Para. 5 P. 927, Paras. 1, 3, & 5 P. 928, Fig. X-65 Pp. 933-934, Figs. X-68 & X-70 P. 938, Para. 1 P. 948, Para. 3	P. 415, Para. 4 P. 416, Item 2 P. 418, Paras. 1 & 5 P. 436, Para. 1 & Item 1 P. 438, Item 2

A motorist's ability to use highway information from signing and delineation is governed by *information acquisition*, or how well the source can be seen. It is also governed by *information processing*, or the speed and accuracy with which the message content can be understood. When either of these key aspects of driver performance is compromised, the result is delayed decision-making, erratic behavior, and maneuver errors.

Taylor and McGee (1973) investigated driver behavior at exit gore areas to determine the causes and characteristics of erratic maneuvers. Interviews were also conducted with many drivers whose actions at the gore area were indicative of route choice difficulties. Analyses of the patterns of erratic maneuvers (cross-gore markings, cross-gore area, stop in gore, back up, sudden slowing, lane change, swerve, stop on shoulder) and on-site driver interviews were used to determine the causative factors of these maneuvers. The most frequent erratic maneuver was crossing the gore marking, which had a 69-percent relative frequency of occurrence for drivers exiting, and a 61-percent relative frequency of occurrence for drivers traveling through the interchange. Most of the motorists who made erratic maneuvers (77 percent) were unfamiliar with the route on which they were traveling. Interviews with exiting motorists who made erratic maneuvers indicated that more than half of the drivers were not adequately prepared for the exit. These drivers indicated that the signs lacked needed information or that the information was misleading. Interviews with drivers who made erratic maneuvers and continued through indicated that approximately one-half had difficulty identifying their direction. Approximately 35 percent stated that the signing was not clear, 21 percent responded that they could not clearly distinguish the location of the exit ramp, and 34 percent thought the road markings were inadequate.

The following discussion of exit signing issues focuses on the legibility of text, the understandability of diagrammatic guide signs, and the placement of devices to provide needed message redundancy while avoiding information overload.

Prior to the Millennium edition of the *MUTCD*, legibility standards assumed that a 25-mm-(1-in-) tall letter was legible at 15.2 m (50 ft), which roughly corresponds to a visual acuity of 20/25; as documented in the Transportation Research Board's *Special Report 218* (1988), this legibility index value of 0.6 m/mm (50 ft/in) exceeds the visual ability of 30 to 40 percent of drivers who are 65 to 74 years of age, even under favorable contrast conditions. The *MUTCD* (2000) section 2A.14 provides guidance for determining sign letter heights, indicating that sign letter heights should be determined based on 25 mm (1 in) of letter height per 12 m (40 ft) of legibility distance. A 0.48-m/mm (40-ft/in) standard for signs can accommodate the majority of older drivers *if* contrast ratios (between the legend and background) are greater than 5:1 (slightly higher for guide signs) and luminance is greater than 10 cd/m^2 (candelas per square meter) for partially reflectorized signs. However, a more conservative standard corresponding to 20/40 vision (i.e., a legibility index of 0.36 m/mm [30 ft/in]) would accommodate a greater proportion of seniors under a wider range of viewing conditions.

Nighttime legibility requirements were addressed by Staplin, Lococo, and Sim (1990), who conducted a laboratory simulation using 28 young/middle-aged subjects (ages 19–49) and 30 older subjects (ages 65–80) to measure age-related differences in drivers' abilities to read unique word combinations (of four letters) on green-and-white guide signs. As expected, older drivers required significantly larger letter sizes to read the (unfamiliar) words than younger drivers. Translating the 6-m (20-ft) subject-to-stimulus distance in the laboratory to a requirement of 183 m (600 ft) to read a freeway sign, the data showed that older subjects would require a letter height of 600 mm (24 in), corresponding to an acuity of 20/46. This corresponds to a legibility index of 0.3 m/mm (25 ft/in) for positive-contrast (lighter characters on darker background) highway guide signs.

In a review of State practices, McGee (1991) reported that Oregon reduced the size of letters on their freeway signs from 333-mm (13.33-in) uppercase and 250-mm (10-in) lowercase to 200 mm (8 in) and 150 mm (6 in), respectively. They received numerous complaints that the signs were difficult to read at highway speeds and they therefore returned the letter sizes to their original heights (George, 1987). By contrast, North Carolina, in consideration of older driver needs, increased the Interstate shield size from 900 to 1200 mm (36 in to 48 in), the uppercase letter size from 400 mm (16 in) to 500 mm (20 in), and the lowercase letter size from 300 mm (12 in) to 375 mm (15 in) on guide signs at freeway-to-freeway interchanges (McGee, 1991). No evaluation was planned to determine the effectiveness of this countermeasure.

Garvey, Pietrucha, and Meeker (1997, 1998) conducted daytime and nighttime controlled field studies with older drivers to compare word legibility and word recognition distances obtained with the Standard Series E(M) font and a new font with the proprietary name Clearview™, which was designed to reduce the effects of a phenomenon referred to as "irradiation," "halation," or "overglow." This phenomenon occurs when bright bold strokes bleed into a character's open spaces under headlight illumination, causing the lettering to appear blurry, rendering the text illegible. The study details are presented on p. 126 in chapter I, Design Element J (Street-Name Signing). Two versions of this experimental font were employed: one version matched Series E(M) in letter width and height, but because of its smaller intercharacter spacing, it resulted in a smaller word area (referred to in this Handbook as Clear 100); and another version contained letters that were increased in size to 112 percent of the standard font, so that the words created with the experimental font (referred to in this Handbook as Clear 112) were the same size as the standard-font words. The signs were erected with a lateral offset of 3.7 m (12 ft) from the center of the observation vehicle (or 1.8 m [6 ft]) outside of the right edge line, and were raised to a height of 1.8 m (6 ft) above the ground. Study results indicated that during the day, the Series E(M) fonts and both of the experimental fonts produced approximately equal reading distances. At night, using low-beam headlights and bright signing materials (see p. 126), the Clear 112 font (occupying as much sign space as the standard font) produced significantly longer legibility distances (22 percent longer) and recognition distances (16-percent increase) than the Series E(M) font. With highway-size signs on 88-km/h (55-mi/h) roadways, this increased legibility could add up to 2 full seconds, or an additional 49 m (160 ft), to the interval in which drivers must read and respond to a sign.

Hawkins, Picha, Wooldridge, Green, and Brinkmeyer (1999) conducted a study of the legibility of three sign alphabets with lowercase letters: the standard US. DOT/FHWA Series E(M); Transport Medium (the alphabet used in Great Britain for overhead guide signs with positive contrast); and Clearview™ (see above). The British letters were designed in the 1950s to eliminate irradiation when viewed at nighttime using headlights (Hawkins et al., 1999), and consequently a narrower stroke width is used for positive contrast signs (white legends on a blue or green background) than is used for negative contrast signs (black legends on white, yellow, or orange backgrounds). Also, the letters are designed to fit on tiles, which are placed side-by-side to create words, eliminating the need to measure distances between words as is done in the United States. Hawkins et al. indicate that the Clearview™ font retains the visual proportions of the standard FHWA alphabets, but it also incorporates desirable attributes from foreign and domestic typefaces, particularly British Transport Medium. However, the Clearview™ letter is typically wider than the same letter in Transport Medium.

Hawkins et al. (1999) employed 54 individuals to participate in the controlled field study as follows: 7 young drivers, age 35 or younger; 18 young-old drivers, ages 55 to 64; and 29 old-old drivers age 65 and older. Word legibility and word recognition data were recorded as an experimenter drove three subjects at a time toward the test signs, during daytime and nighttime. Each test sign was created using three, six-letter words arranged vertically on the sign, all in the same font, using high-intensity white letters on green high-intensity sheeting. The signs measured 3.66 x 2.74 m (12 x 9 ft) and were either ground mounted or mounted overhead. The ground-mounted signs were erected at a height of 2.1 m (7 ft) from the ground to the bottom of the sign, and were placed 9.2 m (30 ft) from the right edge of the travel lane. The overhead signs were erected 6.1 m (20 ft) above the traveled lane. Each word used a 406-mm (16-in) initial uppercase letter, followed by five lowercase letters. The lowercase letter size varied somewhat according to the alphabet type, but was generally 75 percent of the uppercase letter size (305 mm [12 in]).

Results indicated that the Clearview™ font was more legible at both the mean and 85th percentile levels than the Series E(M) font for signs placed overhead, both under daytime and nighttime conditions. The 85th percentile daytime legibility index for the young-old drivers was 0.48 m/mm (40 ft/in), and for the old-old drivers it was 0.36 m/mm (30 ft/in) for the Series E(M) font. The extent of the improvement was in the range of 2 percent; however, some driver groups experienced improvements in legibility distance for the Clearview™ font that were more than 9 percent greater than those experienced with the Series E(M) font. Hawkins et al. state that the improvement was greatest for drivers with poor vision (worse than 20/40). For the ground-mounted signs, Clearview™ was less legible under daytime conditions than Series E(M), and only slightly (less than 2 percent) more legible at night. The Transport Medium font did not show improvements in legibility distance over the Series E(M) font.

In terms of recognition distance, for the overhead signs, the Clearview™ font produced larger recognition distances than the Series E(M) font, in both daytime and nighttime conditions, except for the drivers with minimum (20/40) vision at night. The extent of the improvement in recognition distance was up to 8.7 percent—considerably higher than the improvements found in legibility distance, and translates to an increase in recognition distance of up to 15 m (50 ft). For ground-mounted signs, Clearview™ produced a small improvement at night among worst-case drivers, but showed no improvement under daytime conditions. The 85th percentile recognition distance for drivers age 65 and older for ground-mounted signs at night with Clearview™ font was 8.6 percent greater than that obtained with the Series E(M) font. Under a few conditions (85th percentile daytime distance for ground-mounted signs for all subjects and for those age 65+), the Transport Medium font produced recognition distances that were, on average, 3 percent greater than those obtained with the Series E(M) font. At nighttime, the 85th percentile recognition distance for drivers age 65 and older was 12.2 percent higher for Transport Medium font on the ground-mounted signs than for the Series E(M) font. The authors note that the recognition data showed significant variability from one condition to the next and they caution readers to take care in applying the results.

Overall, Hawkins et al. (1999) state that the results of their study indicate that the Clearview™ font was more effective than the Series E(M) font in the overhead position both during the day and at night, with the greatest improvement achieved for the worst-case drivers. They further indicate that while the Clearview™ font may be more appropriate than the Series E(M) font for overhead signs, ground-mounted signs should continue to use only the Series E(M) alphabet. Hawkins et al. suggest using different design parameters for overhead and ground-mounted signs to account for the differences in performance characteristics of each. They also note that the Clearview™ font is an evolving font, and that there are differences in the font used in the Garvey et al. (1997) study and in current research.

In Knoblauch, Nitzburg, and Seifert's (1997) focus group discussions with older drivers, participants indicated that they prefer overhead signs to those mounted on the side of the road, stating a need for redundant (overhead) signs to provide advance notice of upcoming exits, including the distances to each, and indicating whether the exit is on the right or left side of the highway. This report, coupled with findings that the Clearview™ font provides greater recognition and legibility distance over the standard Series E(M) font when used on overhead signs under both daytime and nighttime conditions, identifies highway destination signs (D1-D3) placed over the highway and fabricated using the Clearview™ font as the preferred practice to accommodate older drivers. However, until the font undergoes the rule-making process and is approved for use on highway signs, no recommendation can be made in this Handbook regarding the use of Clearview™.

Moving from a consideration of legibility issues to the broader question of how well a motorist can actually use highway sign information, reading time and ease of recall for sign messages deserve attention. "Reading time" is the time it actually takes a driver to read a sign message, contrasted with exposure time or available viewing time, which is the length of time a driver is within the legibility distance of the message. As drivers travel, they must look away from the highway to read signs posted overhead or at the side of the road, and then back to the roadway. During each glance, the maximum amount of text that can be read is three to four *familiar* words or abbreviations. A motorist's rapid understanding and integration of message components in memory will greatly assist his/her recall of the message while deciding upon a response. Two errors in message presentation must be avoided: (1) providing too much information in too short a time, and (2) providing ambiguous information that leaves either the intent of the message or the desired driver response uncertain.

Mace, Hostetter, and Seguin (1967) conducted laboratory, controlled field, and observational field studies to evaluate how information presentation time (the amount of time that a sign is readable to a driver) and information lead distance (the distance from an exit that the advance sign is placed) affect exiting behavior at freeway interchanges. They found that 403 m (0.25 mi) is inadequate for information lead distance and, because there were few differences in driver exiting behavior with information lead distances of 805 m (0.5 mi) and 1610 m (1.0 mi), 805 m (0.5 mi) is optimal. In addition, a viewing time of 5 s was adequate for signs containing one to four pieces of information. Lunenfeld (1993) noted that a driver's short-term memory span is between 0.5 and 2 min, and that drivers may forget advance interchange information messages if the time span between the advance notification and the exit ramp exceeds the memory limit. He advocates the use of repetition for interchange information treatments (multiple/successive signs), which will also aid in situations where a sign is blocked by foliage or trucks.

The effect of diagrammatic signing on driver performance at freeway interchanges was studied by numerous researchers in the early 1970s. Bergen (1970) found that graphic guide signs permitted significantly better route guidance performance than conventional signs on certain interchanges, such as collector-distributor with lane drop and multiple split ramps. In pilot studies conducted in New Jersey, Roberts (1972) found that diagrammatic signs that included lane lines were more effective (resulted in a significant reduction in erratic maneuvers) than conventional signs at the interchange of I-287 and U.S. 22, a complex interchange with both left- and right-side exits. Flener (1972) commented on the difficulty in evaluating the effectiveness of traffic control devices in reducing erratic maneuvers at exit gore areas using before and after designs, due to the "novelty effect." Although Roberts (1972) noted that the change *could* be attributed to the greater attention-getting value of novel signs, it was demonstrated that diagrammatic guide signs provide advance information that is readable at a farther distance than that provided by conventional sign text, as well as information about the number of lanes available for any one movement.

Roberts, Reilly, and Jagannath (1974) studied the effectiveness of diagrammatic versus conventional guide signs in a field study at 10 sites. The results were mixed. Several sites showed a reduction in stopping, backing, or weaving erratic maneuvers after installation of the diagrammatic signs. Some sites showed a reduction in stopping and backing maneuvers, but an increase in weaving maneuvers (or vice versa). Other sites showed no change as a function of sign type. Stopping and backing erratic maneuvers were reduced, however, at 9 of the 10 sites.

Taylor and McGee (1973) noted that the main advantage of diagrammatic signing lies in the ability to provide information regarding the interchange layout prior to the exit area. Sign format, however, remains an issue. Conflicting evidence on the effectiveness of diagrammatic signs was reported by Gordon (1972), who found that conventional signs produced fewer lane-placement errors and errors on exit lanes and were more quickly responded to than experimental diagrammatic signs tested at six interchanges in a laboratory study. At the same time, an analysis of particular diagrammatic designs showed that when a diagrammatic sign provided a single arrow or a forked arrow, reaction time was faster and there were fewer errors compared with the conventional sign. Zajkowski and Nees (1976) studied subject response time and correctness of lane choice as a function of sign type in the laboratory. They found that response times were consistently longer for diagrammatic signs than for conventional signs; however, the difference may have been attributable to an increase in information on diagrammatic signs. There were more correct lane-choice responses for conventional signs, and subjects reported more confidence in their lane-choice decisions and a preference for conventional signs. Mast, Chernisky, and Hooper (1972) found that some drivers may require more time to read and interpret information on diagrammatic signs in comparison with conventional signs, and driver information interpretation time may increase as the graphic component of the sign becomes more complex.

Older drivers participating in focus group discussions have indicated that they prefer large, lighted overhead signs with arrows that indicate the lanes for specific destinations, especially if they are approaching a fork in the road (Knoblauch, Nitzburg, and Seifert, 1997). These older drivers stated that the arrows on the overhead signs do not always point to the correct lane, causing drivers to change lanes needlessly; they also stated a desire to see signs that indicate when two travel lanes bear to the same destination.

Brackett, Huchingson, Trout, and Womack (1992) conducted a survey of 662 drivers in 3 age groups (younger than age 25, ages 25-54, and 55 and older) comparing alternative methods of providing lane assignment information on freeway guide signs. The findings of several comparisons in the research are reported, although no analyses using age as an independent variable were performed. First, when two common routes were displayed side-by-side on an exit guide sign, approximately one-half of the drivers believed that the destinations referred to different routes to be accessed by different lanes (i.e., drivers spatially cluster information with each arrow, assuming that information located on the left side of a sign is associated with an arrow also on the left side, and information on the right side is associated with EXIT ONLY or EXIT ONLY with an arrow). When destinations were arrayed one below another, 85 percent of the drivers understood that they were a common route. Second, white downward arrows used in a side-by-side format with an EXIT ONLY (E11-1) panel to indicate that two lanes could exit were misunderstood by 80 percent of the subjects. Third, 56 percent of drivers misinterpreted the phrase NEXT RIGHT on conventional signs as an indication of a mandatory exit and 30 percent misinterpreted the phrase NEXT LEFT in the same manner when these signs were placed over the right and left lanes, respectively. Fourth, when conventional *MUTCD* diagrammatic signs were compared with modified diagrammatic signs that provided separate arrows for each lane, the modified diagrammatic signs resulted in a 13 to 17 percent greater understanding of when a lane must exit and when an adjacent lane may exit or continue through (two-lane exit with optional lane). When the number of arrow shafts exceeded the number of lanes (for example, when there is an auxiliary exit lane downstream of the overhead sign), less than 30 percent of the respondents understood that there would be an added exit lane downstream on the right. With one arrow per lane, comprehension increased by 28 percent over when there were more arrows than lanes (optional use or added lanes). Figure 18 displays: (a) an example of a conventional diagrammatic sign (from *MUTCD* figure 2E-7) and (b) a modified diagrammatic sign for this exit situation.

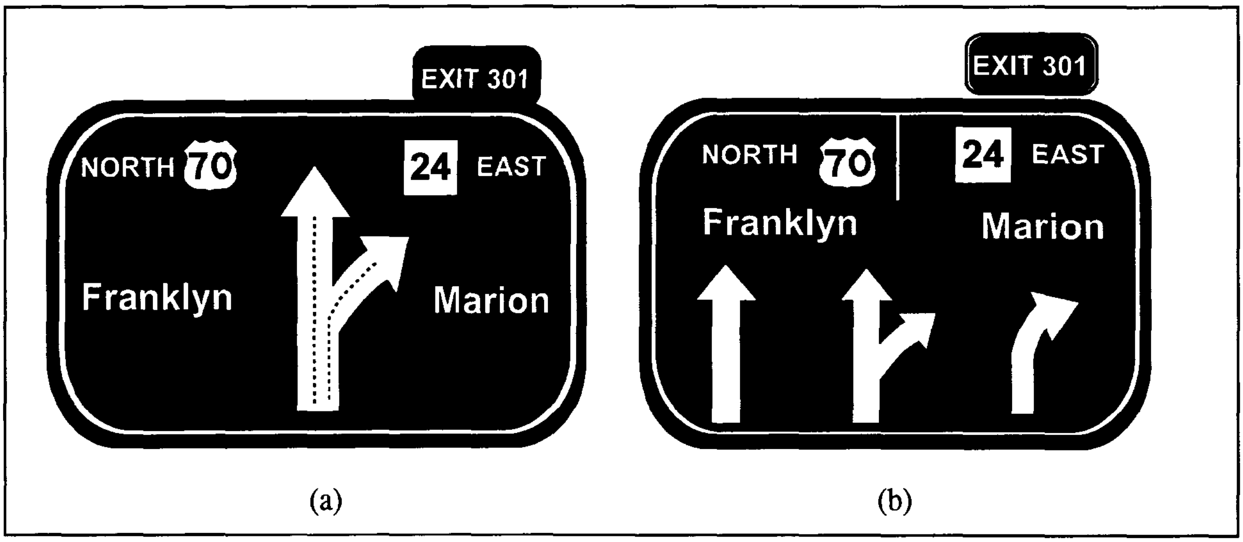

Figure 18. Example of signing used by Brackett, Huchingson, Trout, and Womack (1992) to compare: (a) comprehension of *MUTCD* diagrammatics and (b) modified diagrammatics.

The following discussion of exit-ramp gore delineation focuses on studies conducted to determine which treatments are necessary to ensure rapid and accurate detection of the gore location and ramp heading, particularly under nighttime or reduced-visibility conditions.

Taylor and McGee (1973) reported that the location of the gore is usually perceived easily during daylight hours because a driver can rely on a direct view of the geometry, as well as signing and delineation. However, this task becomes considerably more difficult during darkness, because the driver can no longer rely on a direct view of the geometry, and exit gore signing may be misleading because of the inconsistency in the distance at which it is placed from the nose of the gore area from location to location. At night, delineation is probably the most beneficial information source to the exiting motorist, because it outlines and therefore pinpoints the location of the gore.

Taylor and McGee (1973) measured the effects of the presence of gore area delineation on driver performance at night to determine which of various delineation devices (pavement markings, post delineators, raised pavement markers [RPMs], and a combination of treatments) were most effective. Measures of effectiveness included the point of entry into the deceleration lane, the exiting speed, and any erratic maneuvers. Two right-hand exits, one with a parallel-lane type of deceleration lane and one with a direct-taper type, were selected as test sites. Specifically, the treatment conditions were: (1) post delineator treatment—yellow post delineators placed along the ramp edge of the gore area, plus white delineators positioned along the through side; (2) RPM treatment—yellow RPMs placed on the ramp side of the gore (paint) markings, plus white RPMs on the through side; and (3) combination treatment—the post delineator treatment and the RPM treatment installed in combination.

The baseline condition for this study was moderately worn painted diagonal gore markings and edgelines, with no other delineation devices. All three delineation treatments produced earlier points of entry into the deceleration lane than under the baseline condition. The RPMs were more effective than the post delineators and produced earlier exiting points. The earliest exiting points were found with the combination of RPMs and post delineators. Gore area delineation reduced the frequency of erratic maneuvers at night at both sites. The RPM technique and combination treatment produced significantly lower exiting speeds than did the use of post delineators at one site, and all three treatments produced lower exiting speeds compared with the baseline condition.

The work by Taylor and McGee (1973) also included a comprehensive review of several case studies. As a result of their state-of-the-art summary, coupled with the results of their field observations in the study outlined above, a set of recommendations was developed for pavement marking delineation, post delineators, and RPMs; these recommendations, which have since been widely implemented, are described below.

For pavement marking delineation:

- 200- to 300-mm-(8- to 12-in-) wide white lines should be used to outline the exit gore, and where additional emphasis is necessary, diagonal or chevron markings are recommended.
- A 200-mm-(8-in-) wide line with a 1.5-m (5-ft) mark and 4.5-m (15-ft) gap should be used as an extension of the mainline right edgeline (or median edgeline for left exits) and should replace the lane line for at least 305 m (1000 ft) upstream from the gore nose at an exit lane drop.

For post delineators:

- Post delineators should be placed in the gore area to enhance nighttime visibility. White delineators are recommended for the through-roadway side, and yellow delineators should be used on the exit side. A spacing of 3 to 6 m (10 to 20 ft), depending on ramp divergence angle, is recommended.

- Yellow delineators should be placed along the right edge of the deceleration lane at a spacing of 30.5 m (100 ft). Beyond the beginning of the gore, the spacing is dependent on the degree of curvature.

- White delineators should be placed on the inside shoulder of the through roadway, at a spacing of 30.5 m (100 ft), to help strengthen the through-way delineation in the exit area.

For RPMs:

- Raised pavement markers are recommended as a supplement to standard gore pavement markings and should be placed inside the "V" formed by the pavement marking lines.

- Raised pavement markers should be supplemented with post delineators where the view of the roadway is limited, such as at vertical sections.

Other researchers have also evaluated the effects of RPMs at exit gore locations. RPMs have been shown to reduce erratic maneuvers through (painted) gores at exits and bifurcations. Hostetter, Crowley, Dauber, and Seguin (1989) conducted a controlled field study using 15 subjects ages 18 to 60+ to determine the effect of lighting, weather, and improved delineation on driver performance. Data were obtained on two exits in dry and wet weather under full lighting with baseline delineation (see diagram in Recommendation IIA[4a]). The baseline system is similar to the delineation used at many of the partially lighted interchanges cataloged by the study authors during site selection and, in the opinion of an expert panel convened during the research, constituted a minimum system for partially lighted interchanges. Data were then obtained under partial lighting, with baseline and three improved delineation systems.

Upgrade 1 investigated by Hostetter et al. (1989) differed from the baseline in the use of RPMs along the left ramp stripe and in the substitution of fully retroreflective posts (1150-mm [46-in] strip of 75-mm-[3-in-] wide sheeting) for partially retroreflective posts (450-mm [18-in] strip of 75-mm-[3-in-] wide sheeting) in the physical gore. Upgrade 2 differed from the baseline in the deployment of additional posts along the left ramp shoulder to create a spacing of 15 m (50 ft) rather than 30.5 m (100 ft) and in the installation of wide RPMs (traffic diverters) on the gore strips to replace the 10-mm (4-in) RPMs placed adjacent to the gore stripes in the baseline system. Upgrade 3 replaced all baseline system partially retroreflective posts with fully retroreflective posts except in the gore, used RPMs along the left ramp stripe, and used beaded profiled tape containing a raised-diamond pattern for gore striping. The tape was used because it would project above a film of water during rain. The test sites were a half-diamond interchange and a full diamond, which contained very little ramp curvature. The exit ramps were 4.3 m (14 ft) wide, with a single lane widening to two lanes near the intersection with the crossing roadways. Measures of effectiveness included ramp and spot/trap vehicle speeds, overall travel time, deceleration estimates, and lane placement, as well as selected types of erratic maneuvers and brake and high-beam headlight activations.

Analysis of delineation effects on ramp and spot speeds, and on speed distributions showed few differences under dry conditions. Under rainy conditions, effects were stronger, but were neither large enough nor consistent enough to indicate improved delineation over the baseline system. Although Upgrade 3 produced fewer edgeline encroachments under both dry and wet conditions, from the standpoint of operations, safety benefit, or cost-effectiveness, the upgrade did not demonstrate enough advantage to merit a recommendation for use on diamond interchanges with little ramp curvature.

Lerner, Benel, Huey, and Steinberg (1997) conducted a series of laboratory and field studies to identify conspicuity and comprehensibility problems with current object markers across various hazardous situations for young/middle-aged drivers (ages 20 to 40), young-old drivers (ages 65 to 69), and old-old drivers (age 70 and older). In addition, novel object markers and pavement markings were evaluated to determine whether they improved conspicuity or understanding over the current Types 1, 2, and 3 object markers. With regard to object markers used at gore areas, the *MUTCD* (section 3C.03) states that "In some cases, there may not be a physical object involved [that needs to be marked], but other roadside conditions such as narrow shoulder drop-offs, gores, small islands, and abrupt changes in the roadway alignment may make it undesirable for a driver to leave the roadway. Type 2 or Type 3 object markers may be used at such locations."

In the laboratory experiment conducted by Lerner et al., 64 subjects viewed color photographic illustrations of an object marker situated within a roadway scene. Six stimuli were used to mark gore areas: (1) a Type 1 object marker; (2) a Type 3 object marker; (3) yellow cones; (4) green cones; (5) modified French gore signs (signs used in France, consisting of two white isosceles triangles pointing left and right on a green background, were modified to show black triangles with a cut-out base pointing left and right on a yellow background); and (6) a novel treatment displaying double modified chevron arrows (directional arrows pointing left and right, derived from the chevron alignment sign, consisting of black arrows on a yellow background). The dependent measure was the correctness of response (e.g., the subject correctly identified the hazard or described the correct driving action). Percent-correct ratings for each marker presented in a gore situation across age were as follows: Type 1 (80 percent); Type 3 (75 percent); yellow cones (46.7 percent); green cones (73.3 percent); French gore markers (56.3 percent); and double modified chevrons (87.5 percent). For drivers over age 70, the percent-correct ratings were as follows: Type 1 (83.3 percent); Type 3 (76.8 percent); and double modified chevron (100 percent). One finding of interest that should be highlighted here is the lack of understanding of directional information presented by a solitary Type 3 object marker. In a study conducted during the problem identification stage of this research, participants were correct 39 percent of the time about the direction the marker conveyed (i.e., drive in the direction of the downward pointing stripes).

Next, the object markers were viewed on a test track by different groups of subjects in the same three age ranges to determine daytime and nighttime detection distances. Each trial began at 305 m (1000 ft). An experimenter drove along a test track toward the object marker, with the subject seated in the passenger seat. When a subject was just able to discriminate some feature of the marking, the experimenter was told to stop, and this detection distance was recorded. The experimenter then continued to drive toward the marking, stopping every 15 m (50 ft), at which point the subject described the salient features of the marking. In addition to the markers described above, the following pavement markings were also evaluated: a double edgeline pavement marking and diagonal hash-mark pavement markings. The first finding of interest was that the threshold distances were significantly greater for the post-mounted marker types than for the pavement marking types. There were no main effects of age group or marker type for the pavement markings and no interactions between these variables. The mean nighttime detection distance for the hash marks was 68 m (224 ft) and for the double edgeline it was 69 m (226 ft). By comparison, all of the subjects detected all of the other post-mounted markers at 305 m (1000 ft) at night under low-beam headlight illumination. This finding underscores the importance of including post-mounted markings at gore areas to supplement pavement markings applied in these areas. There was no age group effect or interaction between age group and post-mounted marker type for detection distance.

In terms of nighttime symbol recognition distance, the Type 1 and Type 3 object markers had the highest mean recognition distances (240 m and 277 m [790 ft and 910 ft], respectively). While Type 3 was the single best performer, both had significantly higher mean symbol shape recognition distances than all other markers. Next in terms of nighttime symbol recognition distance were the Type 2 object marker, the double modified chevrons, and the French gore signs, which were not significantly different from each other. Significant main effects of age group and marker type were found, there was no significant interaction between the two.

The only marker that resulted in a significant change in daytime detection distance was the small Type 2 marker, for which detection distance was significantly *reduced* for the age 70 and older group versus the young/middle-aged subjects. The mean detection distance of the Type 2 object marker was 280 m (919 ft) by the young-old drivers and 245 m (803 ft) by the old-old drivers, compared to 305 m (1000 ft) for the young/middle-aged drivers. In fact, all of the subjects in the age 65 to 69 age group saw all other post-mounted markers at 305 m (1000 ft), and the drivers over age 70 saw all other post-mounted markers at distances ranging from 297 to 305 m (973 to 1000 ft). The Type 2 marker also resulted in significantly shorter daytime color recognition distances than any other post-mounted marker type.

Finally, a limited validation study, conducted by Lerner et al. on actual roads in Calvert County, Maryland, compared the Type 1 object marker and the double modified chevron post-mounted markers at a gore situation. The Type 1 object marker produced a correct response rate of 50 percent, compared to 82.4 percent for the double modified chevron design, across driver age groups. However, the double modified chevron marker was better understood than the Type 1 marker at a gore location only during the daytime.

Thus, based on the findings of the laboratory and controlled field studies conducted by Lerner et al. (1996), undelineated gores (i.e., without any object marker) were identified only 1.5 percent of the time by drivers age 75 and older, highlighting the importance of using object markers at such locations. Another finding of importance is that the Type 2 object marker is a poor choice for marking gore areas, particularly for older drivers. The novel double modified chevron marker may be the best candidate for marking gore locations; however, more research would be required to enable a recommendation to be made for its use, based on its poorer nighttime performance compared to the Type 1 marker. Currently, the *MUTCD* only allows for the use of a Type 2 or a Type 3 marker for objects adjacent to the roadway (such as a gore). Based on the poor performance demonstrated by the Type 2 marker in the Lerner et al. research, a recommendation to use the Type 3 marker at freeway gore locations is made in this Handbook. It is also noted that, based on the comprehension data, a Type 1 marker may be a better candidate—for use as an experimental device—when other treatments have not been proven to be successful.

B. Design Element: Acceleration/Deceleration Lane Design Features

Table 31. Cross-references of related entries for acceleration/deceleration lane design features.

Applications in Standard Reference Manuals		
MUTCD (2000)	AASHTO *Green Book* (1994)	*Traffic Engineering Handbook* (1999)
Sects. 2C.10 & 3D.03	Pp. 126-127, Sect. on *Decision Sight Distance* P. 573, Para. 4 Pp. 904-907, Sect. on *Auxiliary Lanes* P. 920, Para. 5 Pp. 937-939, Sects. on *Left-Hand Exits & Traffic Control* Pp. 941-944, Sects. on *Speed-Change Lanes & Single-Lane Free-Flow, Entrances* Pp. 944-961, Portion of Sect. on *Single-Lane Free-Flow Terminals, Exits*	Pp. 319-322, Sect. on *Deceleration and Acceleration Rates* P. 375, Sect. on *Decision Sight Distance (DSD)* P. 393, Item 6

Studies dating back to the 1960s have addressed the effects of ramp design on driving performance; however, Koepke (1993) reported that the basic design criteria, and therefore design standards, used by governmental agencies to design exit and entrance ramp terminals have not changed in more than 30 years. Recommendations for selected design features for interchange ramps may be justified by both the changing characteristics of the driving population and the operating characteristics of the highway system. Age-related functional decreases in visual acuity, motion judgment, and information-processing capabilities cause increased difficulty for older drivers entering and exiting highways. At the same time, traffic density has increased dramatically, resulting in more complex decision-making and divided-attention requirements at these sites. In a survey of 664 drivers age 65 and older, one-half of those surveyed (49 percent) reported that the length of freeway entry lanes was a highway feature that was more important to them now compared with 10 years ago (Benekohal, Resende, Shim, Michaels, and Weeks, 1992).

The difficulties older drivers are likely to experience on freeway ramps, particularly acceleration lanes, are a function of changes in gap judgment ability resulting from a diminished capability to accurately and reliably integrate speed and perceived distance information for moving targets; reduced neck/trunk flexibility; and age-related deficits in attention-sharing capabilities. First, the requirement to yield to approaching traffic on the mainline requires a merging driver to assess the adequacy of gaps in traffic by turning his/her head to look over the shoulder and/or by using the sideview mirrors. In a survey of 297 adults ranging in age from 22 to 92, which was conducted to gain a greater understanding of the visual difficulties they encounter while driving, the older participants reported greater difficulty judging both the speed of their vehicle and the speed of other vehicles, and they expressed a concern over other vehicles "moving too quickly" (Kline, Kline, Fozard, Kosnik, Schieber, and Sekuler, 1992).

It has been shown that older persons require up to twice the rate of movement to perceive that an object is approaching, and require significantly longer to perceive that a vehicle is moving closer at a constant speed, compared with younger individuals (Hills, 1975). Darzentas, McDowell, and Cooper (1980) used Hills' data in a simulation model to estimate conflict involvement for each class of subject as a function of main-road flow and speed. In the model, a conflict occurs when a poor gap acceptance decision is made by a driver, causing an oncoming vehicle to decelerate to avoid a collision. The model indicated that older drivers were involved in more conflicts than younger drivers of the same gender, and male drivers were involved in more conflicts than females in the same age class at all flows.

Other findings describing age differences in driver behavior on acceleration ramps are reported in a National Highway Traffic Safety Administration (NHTSA) study of driver age and mirror use. In this study, which measured the time required to make a "safe/unsafe" maneuver decision in a freeway lane-change situation, old-old drivers (age 75 and older) consistently required longer response times to make a lane-change decision than a group of drivers ages 65–74, who, in turn, demonstrated exaggerated response times compared with a younger control group (Staplin, Lococo, Sim, and Gish, 1996). This was a simulator study, using large screens showing dynamic videos of overtaking vehicles, in correct perspective, as the test stimuli; also, all drivers were forced to rely on their mirror information alone to make the maneuver decision in this research. The mean response time for a lane-change decision for the oldest (age 75 and older) driver group in this study, across a large number of trials in which the relative speed of the overtaking vehicle was varied between 16.1 and 40.25 km/h (10 and 25 mi/h) (i.e., faster than the subject's own vehicle was traveling when the video was shot), changed with changes in the target distance (separation of overtaking vehicle from driver). At close separation distances (30.5 to 61 m [100 to 200 ft]), where virtually all older drivers quickly decided that a lane-change maneuver was unsafe, decision latency averaged approximately 2.1 s. At a 61-m (200-ft) separation distance, some drivers were more willing to merge and required longer to reach a maneuver decision, producing a mean latency of 2.5 s. At a 91.5-m (300-ft) separation distance and greater (between the overtaking vehicle and the driver wishing to change lanes), maneuver decision latency reached an asymptote at 2.95 s, as increasing percentages of subjects accepted the available gap ahead of the overtaking vehicle.

Findings from reviews of crash rates and ramp characteristics are also relevant. Lundy (1967) found that off-ramp crash rates were consistently higher than on-ramp crash rates. However, Oppenlander and Dawson (1970) reported that at urban interchanges, 68 percent of the interchange ramp crashes occurred at entrance ramps, while 32 percent occurred at exit ramps; for rural interchanges, these percentages were reversed. Similarly, Mullins and Keese (1961) reported that in urban areas, 82 percent of the interchange crashes occurred at on-ramps and 18 percent at exit ramps. Furthermore, Lundy's (1967) study of 722 freeway ramps in California found that the crash rate was reduced for off-ramps when deceleration ramps were at least 274 m (900 ft) long (not including the length of the taper), for on-ramps when acceleration lanes were at least 244 m (800 ft) long, and for weaving sections that were at least 244 m (800 ft) long. Oppenlander and Dawson (1970) also concluded that safety was improved for on-ramps, off-ramps, and weaving areas 244 m (800 ft) in length or greater. Cirillo (1970) found that increasing the length of weaving areas reduced crash rates, and increasing the length of acceleration lanes reduced crash rates *if* merging vehicles constituted more than 6 percent of the mainline volume. Reduced crash rates from lengthening of *deceleration* lanes also appears to be related to the percentage of diverging traffic, with significant safety benefits beginning when 6 percent of the mainline traffic diverges (Cirillo, 1970).

The most comprehensive work to develop guidelines for freeway speed-change lanes (SCLs) was conducted in NCHRP project 3-35 by Reilly, Pfefer, Michaels, Polus, and Schoen (1989), who collected data on the entry and exit processes by videotaping 35 sites in three States. An entrance model was developed, based on gap acceptance and acceleration characteristics of drivers as determined by the controlling geometry. An exit model was developed, based on the driver's behavioral response to design geometrics. The purpose of the research was to develop new criteria that would offer greater flexibility than the (then) current AASHTO (1984) guidelines, which "do not provide the designer with the ability to reflect important geometric and traffic conditions" (Reilly et al., 1989). In this research, it was reported that the AASHTO (1984) SCL design criteria were based on the acceleration and deceleration characteristics of early-model vehicles, with little regard to traffic flow characteristics or driver behavior. The design values produced by the NCHRP project entry model for SCL length were slightly lower at low freeway speeds and significantly higher at moderate to high freeway speeds when compared with the 1984 AASHTO values. The exit model values for length were significantly higher than 1984 AASHTO values for all freeway and ramp speeds. The findings of the study suggest that for certain traffic conditions, the current SCL design criteria do not provide sufficient length for proper execution of the merge or diverge process. This is of particular importance with regard to the age-related diminished capabilities documented above.

In the consideration of *acceleration lanes and entrance ramps*, Michaels and Fazio (1989) reported on the model of freeway merging developed during the conduct of NCHRP project 3-35 to define SCL length. In this model, the merge process is composed of four sequential decision components, to which a fifth component is added: (1) a steering control zone (SC), which involves the steering and positioning of the vehicle along a path by steering from the controlling ramp curvature onto the SCL; (2) an initial acceleration zone (IA), in which the driver accelerates to reduce the speed differential between the ramp vehicle and the freeway vehicles to an acceptable level for completing the merge process; (3) a gap search and acceptance zone (GSA), during which the driver searches, evaluates, and accepts or rejects the available lags or gaps in the traffic stream; (4) a merge steering control zone (MSC), during which the driver enters the freeway and positions the vehicle in the nearest mainstream lane (Lane 1); and (5) a visual clear zone (VC), which provides a buffer between the driver and the end of the acceleration lane, where the driver can either merge onto the freeway in a forced maneuver or abort the merge and begin to decelerate at a reasonable rate. Associated with each of these components is a length; the total SCL length is the sum of the SC, IA, GSA, and VC components. The entry process is diagrammed in figure 19.

Design values for entrance ramp acceleration lane lengths were developed as a part of NCHRP project 3-35 based on driver behavior and traffic flow characteristics obtained from field studies and known human factors. The model assumes that a driver will adopt a significant non-zero speed differential at the beginning of the GSA in order to facilitate entry into the traffic stream. In this model, it is recommended that a value of 16.1 km/h (10 mi/h) be used for that speed differential. In this research, it was found that it is not only the speed differential between the ramp and freeway vehicles, but also the position of the vehicles relative to each other and the availability of a suitable gap in the freeway traffic that determine when the merge will occur.

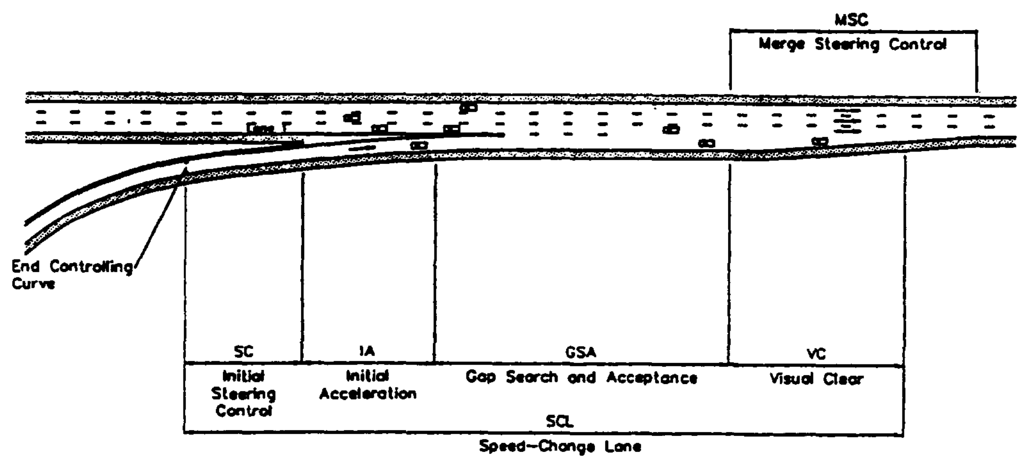

Figure 19. The entry process and components of the entry model developed in NCHRP project 3-35.

The time for the SC is considered to be a constant, which is approximately 1 to 1.5 times the entry velocity, as it was estimated that a 1-s steering transition from ramp to acceleration lane would be sufficient. Therefore, at an entry speed of 15 m/s (50 ft/s), a maximum of 23 m (75 ft) should provide for the entry steering maneuver. The length of the acceleration segment (IA) depends on the magnitude of acceleration that is acceptable to the driver. If the driver accelerates at 1.5 m/s^2 (4.8 ft/s^2) for only 2 s, he or she will have traveled 33.5 m (110 ft), which, when added to the steering control distance, means that the driver will have a clear view of oncoming traffic for a minimum of 49-56 m (160–185 ft). The appropriateness of these model assumptions for older drivers was not addressed in the NCHRP project, however.

As emphasized in NCHRP project 3-35, the GSA is a key component of the entry model; this is especially true for older drivers. This length includes the distance required to search for and accept a headway, and is determined by the distribution of headways in Lane 1 of the freeway, the gap acceptance characteristics of the driver of the ramp vehicle, the design vehicle (car or truck), and the volume on the ramp. A vehicle on the mainline approaching a driver on the freeway ramp appears larger to the merging driver the closer it becomes. The angular velocity is the rate of change in the angle between the approaching vehicle and the merging driver over time. The angular velocity threshold—a critical variable because of its impact on GSA length and overall acceleration lane length—is set at 0.002 rad/s in the entry model. This value is based on field measurements and ensures that 85 percent of observed drivers in model validation studies (age not reported) will accept a gap when the rate of approach of the mainline vehicle translates into an angular velocity consistent with this value. The GSA length requires the use of 16 equations, which are documented in the NCHRP project 3-35 report. There are a number of problems in applying these formulations using an older design driver, however. While it has been reported that drivers accept shorter gaps in freeway traffic than assumed by the model (Koepke, 1993), the critical gap size for this and for other maneuvers increases significantly with increasing driver age. In addition, whereas Michaels and Fazio (1989) cited observed behavior whereby drivers judge gaps in sequence, increasing the probability of finding one acceptable by accelerating between successive searches, there is ample anecdotal evidence of older drivers slowing and often stopping in acceleration lanes when their initial search does not reveal an acceptable gap in which to merge with traffic on the mainline (Transportation Research Board, 1988). Finally, noting the increased reliance on mirror information for gap judgments in this situation by (older) persons with reduced neck/torso mobility, the exaggerated maneuver decision latencies in the Staplin et al. (1996) research on mirror-based lane-change judgments reported earlier bear on GSA (and, therefore, acceleration lane) length requirements.

The VC length is determined such that the pavement area at the end of the ramp taper provides the driver with sufficient distance to implement a forced merge or decelerate to a stop to avoid running off the acceleration lane if he/she has not found an acceptable gap. In the model, if a driver on the acceleration lane is traveling at a speed of 21–24 m/s (70–80 ft/s), then as he/she approaches to within 61–76 m (200–250 ft) of the end of the lane or when the taper produces a lane width of less than 3 m (10 ft), the driver will begin to decelerate. Clearly, the delineation of the pavement width transition at the ramp terminus must be highly conspicuous to accommodate older driver diminished visual capabilities.

Another issue addressed by NCHRP project 3-35 was acceleration lane geometry. Koepke (1993) reported that 34 of the 45 States responding to a survey conducted as a part of NCHRP project 3-35 on SCLs use a parallel design for entrance ramps. Thirty of the agencies interviewed use a taper design for exit ramps and a parallel design for entrance ramps. The parallel design requires a reverse-curve maneuver when merging or diverging, but provides the driver with the ability to obtain a full view of following traffic using the side and rearview mirrors (Koepke, 1993). Although the taper design reduces the amount of driver steering control and fits the direct path preferred by most drivers on *exit* ramps, the taper design used on entrance ramps requires multi-task performance as the driver shifts between accelerating, searching for an acceptable gap, and steering along the lane. Reilly et al. (1989) pointed out that the taper design for entrance lanes poses an inherent difficulty for the driver and is associated with more frequent forced merges than the parallel design. Forced merges were defined as any merge that resulted in the braking of lagging vehicles in Lane 1, or relatively quick lane changes by lagging vehicles from Lane 1 to a lane to the left. The parallel design would thus appear to offer strong advantages in the accommodation of older driver diminished capabilities.

In the consideration of *deceleration lanes and exit ramps*, Livneh, Polus, and Factor (1988) reported that studies analyzing traffic behavior on deceleration lanes have been few in number. They summarized Fukutome and Moskowitz's (1963) efforts to determine whether the length of the ramp tangent approaching the ramp curve had any effect on ramp speed. Fukutome and Moskowitz (1963) found that the length of the deceleration lane from the end of the taper should be at least 137 m (450 ft) when the ramp curve has a radius of 122 m (400 ft), and noted that shorter distances resulted in significantly lower speeds at the nose, which were reflected backward, causing interference to through traffic on the freeway. The results suggested that the shorter distances resulted in unnaturally high rates of deceleration, primarily affecting unfamiliar drivers who are more likely to have adjustment problems when unusual deceleration rates are applied. Fukutome and Moskowitz (1963) found that drivers prefer some moderate deceleration rate as opposed to an extremely low one afforded by a lengthy distance in which to accomplish the speed change. The design should allow the vehicle to enter the deceleration lane at a speed comparable to the speed of through traffic and decelerate in the deceleration area to the velocity required to negotiate the exit ramp properly.

As in the case of acceleration lanes, the speed-change maneuver on deceleration lanes was segmented into components in NCHRP project 3-35 (Reilly et al., 1989). These components include: (1) the *diverge steering zone*, L_{DS}, which is the distance upstream from the exit gore at which a driver begins to diverge from the freeway; (2) the *steering control zone,* L_{SC}, in which the driver steers and positions a vehicle from the freeway lane onto the deceleration lane; (3) the *deceleration in-gear zone*, L_{DG}, in which the vehicle decelerates prior to braking; and (4) the *deceleration while braking zone,* L_{DB}, in which braking occurs in order to reach a reduced speed dictated by the geometrics, terminus, or traffic conditions on the off-ramp. The total deceleration lane length, L_{SCL}, is equal to $L_{SC} + L_{DG} + L_{DB}$. Figure 20 diagrams the exit process defined in the NCHRP research. The lengths of the four zones in the exit process were combined into two design elements: the L_{SCL}, which is the total length required to complete the exit process, and the L_{DS}, which defines the distance upstream from the nose of the exit wedge at which the beginning of the deceleration lane must be placed. Depending on the location of the speed-controlling point on the ramp, which could be controlling curvature or a queue of stopped vehicles, the driver will decelerate in gear until the perceived rate of approach to this point dictates the need to brake. Therefore, the total deceleration of the vehicle is a combined process between in gear and braking. The length of the L_{DG} zone is the most sensitive to variations in diverge speeds; the L_{SC} and L_{DB} zones vary little with diverge speed. The design criteria for deceleration lanes are presented in *NCHRP 3-35 Speed-Change Lanes User Design Guidelines*; these criteria can be used to determine the required lengths for a new design, to test the appropriateness of an existing design, or to retrofit older designs not used by designers today.

A comparison of the values generated by the NCHRP exit model and current AASHTO values was presented by Reilly et al. (1989). For most freeway and ramp speeds, the model deceleration lane lengths are longer than the AASHTO values. The difference between the exit model and AASHTO values increases with increasing ramp speed. The NCHRP model was validated using data observed at 12 sites. An assumption in the development of the exit model was that the speed of an exiting vehicle during the diverge steering maneuver is constant and, therefore, the speed of the vehicle during the diverge equals the freeway speed. Data collected

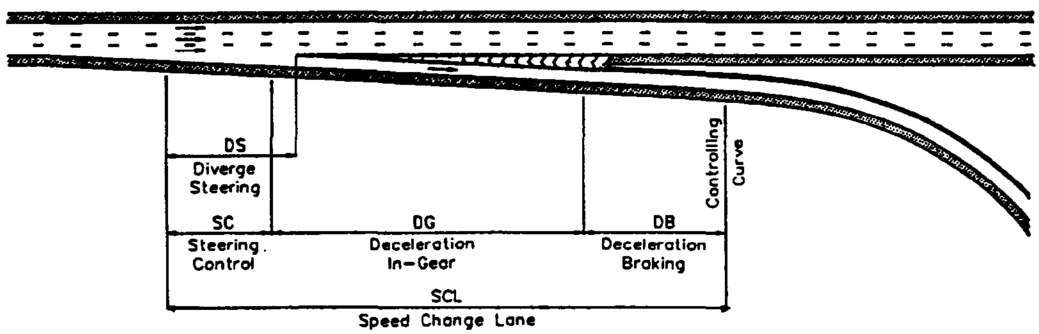

Figure 20. The exit process and components of the exit model developed in NCHRP project 3-35.

at 12 exiting sites during this study confirmed that the reduction in speed was normally less than 3.2 km/h (2 mi/h), regardless of the initial speed. However, it was found that a significant percentage of drivers reduce their speed while still on the freeway, prior to the diverge maneuver, with an average speed of 83.7 km/h (52 mi/h) across all sites prior to the diverge maneuver. Next, a critical element in the exit model is the angular velocity threshold, which determines L_{DS} and L_{DB}. As a driver approaches an exit, he/she first recognizes the taper diverging from the freeway lane, which is essentially a widening of the overall roadway. This recognition is determined mainly by the change in the driver's visual angle subtended by the roadway; however, other elements such as edge markings and signing will generate a component of angular velocity. In addition, the angular velocity will reach threshold at greater distances for a curved ramp than for a simple diverging ramp, resulting in the use of more deceleration lane length in cloverleaf interchanges than in diamond interchanges.

Complementing the findings in NCHRP project 3-35, Livneh et al. (1988) observed traffic using freeway deceleration lanes at two freeway sites to record actual behavior and compare it to current design practices. They concluded that a considerable difference exists between the AASHTO assumptions and actual driver behavior along deceleration lanes. The principal discrepancies were in average speeds and in rate and duration of deceleration in gear and while braking. The speed of both cars and heavy vehicles at the beginning of the deceleration lane was always lower than the average speed of through vehicles. The deceleration values obtained were lower than the values recommended by AASHTO. On properly designed long lanes, the duration and length of deceleration in gear were longer than 3 s, as assumed by AASHTO, and deceleration in gear took place for an average of 10 s until the speeds of the vehicles slowed from about 85 percent of their average running speed on the through lane–the initial speed at the beginning of the taper–to an average of 67 percent. From this point, which was 200 m (650 ft) from the beginning of the deceleration lane, braking started and continued until speeds were further reduced to meet the average running speed required to safely negotiate the ramp curve that followed.

To meet the needs of older drivers, the point of controlling curvature on an exit ramp, as well as the curve speed advisory, must be highly conspicuous to create an appropriate expectancy of the required vehicle control actions. With this expectancy, older drivers should be able to competently negotiate deceleration lane geometries meeting AASHTO or NCHRP guidelines (also assuming effective gore delineation as discussed in Handbook Section II-A). Raised curve delineation treatments are recommended in this regard; post-mounted delineators or chevrons are particularly effective (see Handbook Section III-A). In addition, Holzmann and Marek (1993) noted that ramp operations may be improved by moving the relatively sharp ramp curvature away from the ramp terminal.

Finally, a recent review of interchange design issues, necessitated by changes in road user characteristics and current research, approached ramp geometry as a three-dimensional system (Keller, 1993). According to this review, the factors that influence ramp alignment and superelevation design include design consistency and simplicity, the roadway user, design speed, and (stopping and decision) sight distances. Because driver reaction time is slowed when elements of ramp geometry are different than expected, design should provide for long sight distances; careful coordination between horizontal and vertical alignment; generous curve radii; and smooth coordinated transitions, particularly when complex interchange designs are unavoidable. Increasing the sight distance and simplifying interchange layout can reduce some of the effects of decreasing visual acuity, short-term memory decline, reduced decision-making ability, reduced ability to judge vehicle speed, decreased muscle flexibility and pain associated with arthritis, and early fatigue and slower reaction times associated with increasing driver age. With regard to design speed, Keller (1993) stated that the ramp proper should be viewed as a transition area with a design speed equal to the speed of the higher speed terminal wherever feasible, and that few diagonal or loop ramps are long enough to accommodate more than two design speeds. Thus, the terminals and the ramp proper should be evaluated to determine the appropriate speed for design.

In terms of stopping sight distance (SSD) requirements, Keller (1993) noted that designers can reduce drivers' stress at interchanges by providing sight distances greater than the minimum SSDs. Although a brake reaction time of 2.5 s is representative of 90 percent of the drivers used in a 1971 study by Johansson and Rumar and is used in the AASHTO SSD formula, it has been suggested that a 3.5-s perception and braking time should be used to accommodate the elderly with diminished visual, cognitive, and psychomotor capabilities (Gordon, McGee, and Hooper, 1984). Another assumption in the AASHTO calculations for SSD is a driver eye height of 1.06 m (3.5 ft); the eye height of older drivers is often less. Finally, alignment affects braking distance such that curves impose greater demands on tire friction than tangents, resulting in increased braking distance when the friction requirements of curves and braking are combined (Glennon, Neuman, and Leisch, 1985).

Keller (1993) noted that locations where SSD values do not provide the time necessary to process information and react properly highlight the importance of the use of decision sight distance (DSD). Examples of locations at interchange ramps where DSD is desirable include ramp terminals at the main road, especially at an exit terminal beyond the grade separation and at left exits; ramp terminals at the cross road; lane drops; and abrupt or unusual alignment changes. AASHTO guidelines (1994) note that sight distance along a ramp should be at least as great as the safe stopping distance. The sight distance on a freeway preceding the approach nose of an exit ramp should exceed the minimum stopping distance for the through traffic speed, desirably by 25 percent or more, although the desirable goal remains DSD.

DSD values–which include detection, recognition, decision, and response initiation and maneuver times–are provided in AASHTO (1994) table III-3 by design speed and type of avoidance maneuver required. Lerner, Huey, McGee, and Sullivan (1995) measured DSD for three driver age groups (ages 20–40, ages 65–69, and age 70 and older) at six freeway lane-drop locations. While perception-reaction time values measured by Lerner et al. (1995) were actually somewhat lower than the values assumed by AASHTO, they nevertheless found that the 85th percentile *total* time required by each age group for detection, decision, and maneuvering exceeded the recommended AASHTO value of 14.5 s. The freeway total times averaged 16.5 s, 17.6 s, and 18.8 s for the three groups (from youngest to oldest), respectively. The researchers explained that the original AASHTO work assumed free-flow traffic conditions in which drivers were not required to wait for a gap in traffic to change lanes. The Lerner et al. (1995) study, by comparison, was conducted on heavily traveled urban freeways, and subjects often had to wait for gaps in traffic before maneuvering. This led to significantly higher maneuver times than were assumed by AASHTO. No modifications to the existing DSD standards were deemed necessary. Keller (1993), reporting on the results of a 1991 survey about distances used when locating ramp exits beyond a crest vertical curve, indicated that 15 (38 percent) of State design agencies use the safe SSD, 9 (23 percent) use the safe SSD plus 25 percent, and 12 (31 percent) use DSD.

C. Design Element: Fixed Lighting Installations

Table 32. Cross-references of related entries for fixed lighting installations.

Applications in Standard Reference Manuals		
MUTCD (2000)	AASHTO *Green Book* (1994)	*Roadway Lighting Handbook* (1978)
Sect. 1A.13, *Sign Illumination* Sects. 2E.05 & 3G.04 Sect. 4B.04, Item H Sect. 6G.13	P. 310, Para. 2 P. 567, Para. 1 & Fig. VIII-6 on p. 570	Pp.14-15, Sects. on *Complete Interchange Lighting & Partial Interchange Lighting* Pp. 16-26, Sects. on *Analytical Approach to Illumination Warrants & Informational Needs Approach to Warrants* Pp. 42-45, Sect. on *Summary of Light Sources* P. 71, 4th bullet Pp. 84-89, Sect. on *Interchange Lighting* Pp. 120-129, Sect. on *Illumination Design Procedure*

Research has documented that: (1) freeway interchanges experience a higher crash rate than the mainline (Cirillo, 1968); and (2) urban freeway lighting has beneficial safety effects (Box, 1972). Cirillo (1968) also found a reduction in the number of interchange crashes as lighting intensity increased. Gramza, Hall, and Sampson (1980) evaluated the interchanges in the Interstate Accident Research (ISAR-2) database at which lighting had been introduced during the 10-year study period. During the daytime, there were 83 crashes before lighting and 80 crashes after lighting. At nighttime, by comparison, there were 76 crashes before lighting and 43 crashes after lighting. Taylor and McGee (1973) found a reduction in erratic maneuvers at exit lane-drop sites in a before-and-after study when the exit area was illuminated during the "after" period of data collection.

Although nighttime driving is associated with a higher crash risk for drivers of all ages, the effects of aging on the visual system are further compounded by the effects of darkness. The aging process causes gradual declines in a variety of visual functions, including acuity, contrast sensitivity, glare recovery, and peripheral vision, making night driving especially difficult for older drivers. Of particular difficulty is the ability to notice and recognize objects at night and in low-light conditions such as dawn and dusk, rain, fog, haze, and snow. Between age 20 and age 70, aging directly reduces contrast sensitivity by a factor of about 3.0 (Blackwell and Blackwell, 1971); older drivers are thus at a greater relative disadvantage at lower luminance levels than younger drivers.

The impact on the older driver of lost sensitivity under nighttime conditions should be assessed against the nature of the night-driving task. Even at night, most visual information is processed by the cone or daylight system in the foveal region of the retina where fine detail is resolved. Artificial lighting raises the illumination level of the roadway environment to the photopic range so that reading and tracking functions can occur. The peripheral rod system participates primarily by alerting the driver to a weaker signal away from the foveal line of sight, which may then be oriented by the driver with a foveal fixation. The implication of a loss in rod sensitivity is that a much brighter peripheral signal will be needed to elicit proper visual attention from the driver, and that signals now falling below threshold will be ignored. In fact, the signal may need to be 10 times to as much as 100 times brighter, depending on age and object color (Staplin, Lococo, and Sim, 1990). Since both rod and cone thresholds increase with age, it is also true that more light will be needed to bring important tasks such as reading and tracking (path maintenance) above the cone limit. In a survey of 1,392 older drivers ages 50 to 97, 70 percent indicated that more highway lighting is needed on freeways. These respondents identified the following areas where more lighting is needed: interchanges, construction zones, and toll plazas (Knoblauch, Nitzburg, and Seifert, 1997).

There are a number of other aspects of vision and visual attention that relate to driving. In particular, saccadic fixation, useful field of view, detection of motion in depth, and detection of angular movement have been shown to be correlated with driving performance (see Bailey and Sheedy, 1988, for a review). While these visual functions do not appear to have strong implications for highway lighting practice, it could be advantageous to provide wider angle lighting coverage to increase the total field of view of older drivers. High-mast lighting systems can increase the field of view from 30 degrees (provided by conventional fixtures) to about 105 degrees (Hans, 1993). Such wide angles of coverage provided by high-mast lighting might have advantages for older drivers in terms of peripheral object detection, thus easing the task of identifying ramp geometry, traffic control devices, and traffic patterns. However, while effective high-mast systems have been demonstrated (Ketvirtis and Moonah, 1995), such installations also tend to sacrifice target contrast for the increased field of view that they provide.

Hans (1993) defines "high mast" as any lighting structure that rises at least 18 m (60 ft) above road level. Some designs extend up to 46 m (150 ft) and higher above the ground. One pole, anchored 15 to 21 m (50 to 70 ft) from the edge of the roadway, may be used to support a cluster of 3 to 12 luminaires. As a comparison, conventional cobra-head poles mounted on the shoulder support one or two luminaires at a height of 8 to 15 m (26 to 50 ft) above the road. For example, the New Jersey *Roadway Design Manual* defines their high-mast lighting system as one that utilizes a mounting height of 30.5 m (100 ft) with a cluster of a maximum of eight, 400-W high-pressure sodium luminaires, and their conventional lighting system as one that utilizes mounting heights of 7.9 m (26 ft) with 150-W high-pressure sodium luminaires for ramp application. The New Jersey Manual states that tower lighting (high mast) shall be considered first (over conventional lighting) for full-interchange lighting, preferably using 400-W cutoff-type luminaires; however, non-cutoff luminaires may be employed if the designer can justify their use.

The following paragraphs summarize studies that: (1) evaluated the effects of lighting on crash experience at interchanges; and (2) evaluated specific aspects of driver performance as a function of number and type of luminaires at an interchange.

Gramza et al. (1980) conducted a crash analysis of 400 nighttime crashes that occurred at 116 interchanges during the period of 1971–1976 in 5 States (Maine, Maryland, Minnesota, Texas, and Utah). In an analysis of the presence of high-mast lighting at interchanges versus no lighting or other kinds of interchange lighting, the presence of high-mast lighting was found to significantly reduce total crash rates, total crashes involving fatalities and injuries, and crashes involving fatalities and injuries other than the vehicle-to-vehicle and vehicle-to-fixed-object categories (e.g., crashes caused by striking pedestrians). Table 33, taken from Gramza et al. (1980), shows the predicted effect of high-mast lighting on annual number of crashes.

Gramza et al. (1980) also found that although the number of lights at an interchange and the level of illumination had no significant effect on the *total* number of nighttime crashes, significant decreases in a variety of distinct crash types were found with increases in illumination. Increases in the illumination level—measured in lux or horizontal footcandles (hfc)—at interchanges were associated with significant reductions in two types of crashes: vehicle-to-fixed-object crashes involving property damage, and vehicle-to-vehicle crashes involving fatalities and injuries. In addition, increases in the number of lights active at an interchange were found to significantly influence (reduce) the following two crash types: vehicle-to-fixed-object crashes involving fatalities and other injuries, and other property damage crashes. The number of lights at an interchange ranged from 0 to 114, with an average of 16 active lights and a median of 10. Thirty-two percent of the interchanges were unlit. As lighting levels increased, crash rates decreased. Illumination ranged from 0.0 lux to 10.76 lux (0.0 hfc to 1.0 hfc), with an average of 5.49 lux (0.51 hfc) for the lighted sections. These four crash types accounted for 61 percent of the crashes observed in the sample.

Table 33. Relative annual effect of lighting type on total nighttime crashes ($n = 400$) at urban and non-urban interchanges. Source: Gramza, Hall, and Sampson (1980).

Night Traffic Volume	Urban			Non-Urban		
	Non-High Mast	High Mast	% Decrease	Non-High Mast	High Mast	% Decrease
5,000	2.0	0.0	100	3.6	0.4	89
7,500	3.8	0.6	84	5.4	2.2	59
10,000	5.7	2.5	56	7.3	4.1	44

Since there were relatively few crashes per interchange per year, Gramza et al. (1980) employed a model to predict the number of each crash type per year, assuming three levels of traffic volume (average nighttime traffic of 5,000, 7,500, and 10,000 vehicles) at partial cloverleaf and other types of interchanges, and allowing varying levels of illumination or varying numbers of lights. The predicted relationships between traffic volume, lighting, and crash frequency showed that reductions in the number of lights and in the level of illumination (hfc) resulted in higher frequencies of vehicle-to-fixed-object and other property damage crashes for all traffic volumes. Vehicle-to-vehicle crashes were also shown to increase in frequency for all interchange types as illumination was reduced.

In addition, the findings at the level of one interchange were translated to estimate, as an overall annual impact for the five-State sample, the relative influence of the lighting variables on the number of crashes at interchanges through three levels of night-traffic volume. A level of 7.53 lux (0.7 hfc) was used to represent the allowable base of average maintained illumination.

Overall, the model predicted that reductions in the level of illumination appear to cause greater increases in the number of crashes than do reductions in the number of lights (Gramza et al., 1980).

Although the work of Gramza et al. (1980) is noteworthy in its attempt to quantify the complex relationships between interchange lighting and safety, it is critical to remember that their model was applied to data derived to fit 1975 conditions—including, by implication, both the then-current number of older drivers *and* their exposure to this highway feature during nighttime operations. By contrast, present and anticipated future driving patterns of older drivers—whose actual numbers, as well as their percentage of all drivers, will increase dramatically—show much higher use rates for freeways (Lerner and Ratté, 1991). This trend should sharply accentuate the safety impacts cited by Gramza et al.

Janoff, Freedman, and Decina (1982) conducted a study to determine the effectiveness of partial lighting of interchanges, where partial interchange lighting (PIL) was defined as lighting that consists of a few luminaires located in the general areas where entrance and exit ramps connect with the through-traffic lanes of the freeway (between the gore and the end of the acceleration ramp/beginning of the deceleration ramp). A complete interchange lighting (CIL) system includes lighting on both the acceleration and deceleration areas plus the ramps through the terminus. In their survey of approximately 50 agencies that supplied information on more than 14,000 interchanges and more than 7,500 interchange lighting systems, it was found that 37 percent of the interchange lighting was CIL and 63 percent was PIL. An observational field study was conducted to determine the effects of lighting level (various levels of PIL, CIL, no lighting, and daylight), geometry of the interchange (straight versus curved ramps), and presence of weaving area versus no weaving area on driver behavior and traffic operations. PIL was stratified by the number of lights at each ramp, and included three levels: PIL 1 (one light), PIL 2 (two lights), and PIL 4 (four lights). CIL test sites included a full cloverleaf in suburban Baltimore, MD, and a three-leg interchange in suburban Philadelphia, PA with luminaire mounting heights of 12.2 and 9.5 m (40 and 31 ft), respectively. The dependent measures included speed and acceleration of individual vehicles traversing the interchanges; merge and diverge points of individual vehicles entering the main road or leaving it; and erratic maneuvers such as brake activation, use of high beams, and gore or shoulder encroachments.

Both field studies indicated that CIL provided a better traffic operating environment than did PIL and that any interchange lighting performed better than no lighting (although the differences were not always as great as between CIL and PIL). In particular, to the extent that traffic flow and safety are important issues, the Janoff et al. study concluded that existing CIL systems should not be reduced to PIL systems. When installing new lighting, if economics are not an overriding issue, a CIL system is preferred over a PIL system. However, a PIL system with one or two luminaires per ramp will normally perform better than no lighting at far lower cost than a CIL system. PIL systems with fewer luminaires (one or two) frequently performed better than PIL systems with a greater number of luminaires (four). This was explained by the fact that drivers may experience transitional visibility problems under the PIL conditions when they are forced to drive from dark to light to dark areas and at the same time perform complex maneuvers such as diverging, merging, and tracking a 90-degree curve.

Hostetter, Crowley, Dauber, and Seguin (1989) noted that when luminaires are not placed downstream of the physical gore of a partially lighted exit ramp, a driver proceeds from a lighted area to a non-lighted area. Citing evidence from various researchers (Boynton and Miller, 1963; Boynton, 1967; Boynton, Rinalducci, and Sternheim, 1969; Boynton, Corwin, and Sternheim, 1970; Rinalducci and Beare, 1974; and Fredericksen and Rotne, 1978), they reported that the effect of going from higher to lower levels of luminance results in a reduction in visual sensitivity, which would help explain the findings of Janoff et al. (1982) that performance under partial lighting was better with fewer luminaires.

On a final note, Bjørnskau and Fosser (1996) conducted a before-and-after study on a section of roadway in Norway to record driver behavior as a function of roadway lighting. One interesting finding was that the percentage of older drivers and female drivers increased after the introduction of roadway lighting. Thus, a secondary benefit of roadway lighting (beyond its capability of reducing crashes) is increased mobility and access to goods and services for older drivers.

D. Design Element: Traffic Control Devices for Restricted or Prohibited Movements on Freeways, Expressways, and Ramps

Table 34. Cross-references of related entries for traffic control devices for restricted or prohibited movements on freeways, expressways, and ramps.

Applications in Standard Reference Manuals			
MUTCD (2000)	AASHTO *Green Book* (1994)	NCHRP 279, *Intersection Channelization Design Guide* (1985)	*Traffic Engineering Handbook* (1999)
Sects. 2A.24, 2B.29, & 2B.30 Tables 2B-1 & 2C-1 Sects. 2E.50, 3B.19, 3B.22 Figs. 3B-20, 3B-22, & 3B-23, 2E-34 through & 2E-36	Pp. 914-915, Sect. on *Wrong-Way Entrances*	P. 23, Paras. 1-2	P. 384, Items 1-2 Pp. 419-420, Sect. on *Size* & Tables 12-1 & 12-2 P. 424, Para. 1 Pp. 428-429, Sect. on *Mounting* P. 438, Item 4

It has been reported that out of 100 wrong-way crashes, 62.7 result in an injury or fatality, versus 44.2 out of 100 for all freeway or expressway crashes (Tamburri and Theobald, 1965). These data highlight the fact that wrong-way crashes are more severe than most other types. The most frequent origin of wrong-way incidents, as reported by these authors, was entering the freeway via an off-ramp.

Results of more recent investigations of the wrong-way problem in California indicate that fatal wrong-way crashes as a percentage of all fatal crashes on freeways have decreased substantially in the last 20 years (Copelan, 1989). The actual number of wrong-way fatal crashes was the same in 1987 as it was in 1963 (about 35 per year), despite the fact that freeway travel has increased fivefold; the reduction appears to be related to the countermeasures employed by Caltrans (California Department of Transportation) over the intervening years, including the implementation of guide and wrong-way signs and pavement markings providing better visual cues. Copelan (1989), while noting that half of the wrong-way driving on freeways was from deliberate, illegal U-turns, reported that additional improvements could still significantly reduce wrong-way crashes. In their study of highway information systems, Woods, Rowan, and Johnson (1970) found that motorists frequently experience difficulty in locating entrance ramps to freeways, and drivers were often confused when there were several side roadways intersecting in close proximity to the interchange area. These researchers suggested that more efficient use could be made of "positive" signing techniques in guiding motorists to the freeway entrance ramps and discouraging drivers from possible wrong-way maneuvers.

Early studies found that the rate of wrong-way driving based on vehicle-miles of travel increased with driver age (Tamburri and Theobald, 1965). In their analysis of 1,214 wrong-way driving incidents that occurred over two 9-month periods on California highways, they found a moderate increase in incidents for drivers ages 30–39 and ages 40–49. Over age 60, the incidents rose rapidly; and over age 70, incidents occurred at rates approximately 10 times higher than for drivers ages 16–29. Lew (1971) reported on an analysis of 168 wrong-way crashes by civilians on California freeways in which the age of the wrong-way driver was recorded. While certain age groups (i.e., 30–39, 50–59, and 60–69) were represented to an extent corresponding closely to their proportion of the driving population, other groups, such as those ages 16–19, 40–49, and 70–79, deviated markedly from expectation. Drivers ages 16–19 experienced approximately one-half of the wrong-way crashes expected for their age group; drivers ages 40–49 experienced three-quarters of the rate expected; and drivers ages 70–79 experienced *more than twice* the number of freeway wrong-way crashes that would be expected.

Age-related diminished capabilities contributing to wrong-way movements include the cognitive capabilities of selective attention and divided attention, and the sensory/perceptual capabilities of visual acuity and contrast sensitivity. Selective attention refers to the ability to identify and allocate attention to the most relevant targets in the driving scenario on an instant-to-instant basis, while divided attention refers to the ability to perform multiple tasks simultaneously. Individuals less capable of switching attention, or who switch too slowly, may increase their chances of choosing the wrong response or choosing the correct response too slowly. Treat, Tumbas, McDonald, Shinar, Hume, Mayer, Stansifer, and Castellan (1977) reported that 41 percent of the crashes in which older adults were involved were caused by a failure to recognize hazards and problems, and 18 to 23 percent of their crashes were due to problems with visual search. The selective attention literature generally suggests that for adults of all ages, but particularly for older drivers, the most relevant information must be signaled in a dramatic manner to ensure that it receives a high priority for processing in situations where there is a great deal of complexity at the level of information to be processed.

Older drivers' use of signs designed to control wrong-way movements is affected by their visual performance capabilities. Letter acuity declines during adulthood (Pitts, 1982) and older adults' loss in acuity is accentuated under conditions of low contrast, low luminance, and high visual complexity. A field investigation of the effect of a driver's age on nighttime legibility of highway signs indicated that older subjects perform substantially worse than younger subjects on a nighttime legibility task using a wide range of sign materials (Sivak, Olson, and Pastalan, 1981).

Aside from difficulties in the use of signs, problems for older drivers at interchanges most likely result from (age-related) deficits in spatial vision related to the timely detection and recognition of pavement markings and delineation. Data from a study by Blackwell and Blackwell (1971) show that between age 20 and age 70, aging directly reduces contrast sensitivity by a factor of about 3.0. Mace (1988) stated that age differences in glare sensitivity and restricted peripheral vision, coupled with the process of selective attention, may cause higher conspicuity thresholds for older drivers. Overall, these deficits point to the need for more effective and more conspicuous signing and delineation.

Violations of driver expectancy, use of alcohol, and a reduction in the ability to integrate information from multiple sources to make navigation decisions while concurrently controlling the vehicle may all result in driver confusion at critical decision points, resulting in wrong-way maneuvers. Tamburri and Theobald (1965) found that many older drivers and drinking drivers did not know where their wrong-way movement began (i.e., they could identify neither where the decision point was nor the location of the wrong-way maneuver).

Vaswani (1974) identified specific sources of wrong-way movements where alcohol was believed *not* to be a factor. In this study, exit ramps on partial interchanges generated wrong-way maneuvers because, unlike the ramps on full interchanges that converge with right-hand traffic, the ramps meet the crossroad at about 90 degrees to accommodate both left and right turns. Therefore the wrong-way entries consist of left turns off of the exit ramp into wrong-way traffic on a two-way divided highway, right turns from the divided highway into traffic exiting the ramp, and left turns from the crossroad into the exit ramp. At intersections with four-lane divided highways (divided arterial and primary highways), 45 percent of the wrong-way entries were at their intersections with exit ramps or secondary roads. The wrong-way entries were due to left-turning vehicles making an early left turn rather than turning around the nose of the median. Almost all of these crashes involved sober drivers.

Some ramp designs are more problematic than others. In Tamburri and Theobald's 1965 analysis of 400 wrong-way incidents where entry was made to the freeway via an off-ramp, the trumpet interchange category had the highest wrong-way entry rate, with 14.19 incidents per 100 ramp-years, and the full-cloverleaf interchanges had the lowest wrong-way entry rate, with 2.00 incidents per 100 ramp-years. Parsonson and Marks (1979) also determined that several ramp types were particularly susceptible to wrong-way movements, as follows: half-diamond (3.9 per month), partial-cloverleaf ("parclo") loop ramp (11.0 per month), and parclo AB loop ramp (6.7 per month). The parclo loop ramp and the parclo AB loop ramp share the same problem, which is an entrance and exit ramp in close proximity. The half-diamond is susceptible because it is an incomplete interchange, and drivers may make intentional wrong-way entries. A "problem" ramp has been defined as one that experiences more than five wrong-way movements per month; a corrected ramp has less than two per month (Rinde, 1978).

Preventative measures for reducing the frequency and severity of wrong-way maneuvers include modifications in ramp and roadway geometry, modifications in signing and pavement markings, and the use of warning and detection devices and vehicle arresting systems. Selected countermeasures are discussed below.

Vaswani (1974) found that on almost all the interchanges on which wrong-way entries had been made into the exit ramp or from the exit ramp onto the crossroad, the corner of the exit ramp flared into the right pavement edge of the crossroad. He suggested that such a flare provides for a very easy, but incorrect, right-hand turn, and may help to induce a driver to make a wrong-way entry from the crossroad into the exit lane. A countermeasure consisting of a sharp right-hand junction would require a driver to reduce speed and almost come to a stop before maneuvering into the left lane, and would also reduce the chances that a driver exiting the ramp would turn left into wrong-way traffic on the crossroad. Site inspections showed that where the flare was not provided and the left lane of the exit ramp and the passage through the median were channelized, no wrong-way entry to or egress from the exit ramps was reported. In addition, Vaswani (1974) reported that generous widths of an exit ramp with its junction with the crossroad make wrong-way entry or egress from the exit ramp easy. Narrow pavement widths will discourage such entries. However, serious impediment to turning maneuvers by heavy vehicles could also result from this strategy.

Vaswani (1974) also indicated that too large a set-back of the median noses from the exit ramp increases the width of the crossover and makes the intersection harder to "read." Vaswani suggests that if the width cannot be reduced, then pavement nose markings in the form of a striped median should be applied for improved visibility of this design element. See also the discussion on p. 100 of this Handbook for design element IE, about extending delineation treatments from a set-back median nose to the intersecting roadway.

Campbell and Middlebrooks (1988), following the recommendation of Parsonson and Marks (1979) to widely separate the on- and off-ramps at partial-cloverleaf interchanges, experimented with a design in which close exit and entrance ramps would be combined into one paved surface separated only by a double yellow line. Ten ramps in the Atlanta, GA area were redesigned and evaluated using actual counts of wrong-way movements. Two of the ramps were monitored before and after they were converted to combined ramps. At the first location, the wrong-way rate per month before construction was 86.7; after combining the ramps, the rate fell to 0.3 per month. At the second location, the wrong-way rate was 88.6 per month. After the installation of four countermeasures (trailblazers, lowered DO NOT ENTER and WRONG WAY signs, 450-mm [18-in] stop bar, and 200-mm [8-in] yellow ceramic buttons in the centerline of the crossroad), the rate dropped to 2.0 per month. Once the ramps were combined at this second location, the wrong-way rate jumped to 30.0 per month, even when ceramic buttons, permanent signing, and pavement markings and a dotted channelizing line (i.e., pavement markings that lead turning vehicles onto the ramp) were employed.

The mixed results of the Campbell and Middlebrooks study (1988) led to the evaluation of 15 additional combined ramps in the same research project, 12 of which were partial cloverleaf, with the balance consisting of median entrance/exit ramps (designed for future access by high-occupancy vehicles to the median lanes, but during the study period were open to all traffic). The study periods ranged from 30 to 102 days. The results clearly indicated that the concept of combined exit and entrance ramps can work when signing and markings conform to *MUTCD* specifications. It was recommended that 200-mm (8-in) yellow ceramic buttons be installed along the cross-street centerline if all other countermeasures do not work.

With regard to signing, Woods et al. (1970) indicated that positive signing that indicates the correct path or turning maneuver to the motorist rather than a restriction may help most to minimize driver confusion at freeway interchanges. Examples include route markers, trailblazers, and a FREEWAY ENTRANCE sign that positively designates an entrance to the freeway. Friebele, Messer, and Dudek (1971) noted that the use of oversized signs and reflectorization may be needed in locations where motorists are apt to disregard wrong-way warnings, and Copelan (1989) suggested that the larger, highly retroreflective signs may be helpful for confused or elderly drivers.

Parsonson and Marks (1979) found that lowering the DO NOT ENTER and WRONG WAY signs to 450 mm (18 in) above the pavement to place them in the path of the headlight beams at night and placing trailblazer signs on the on-ramp were effective, inexpensive countermeasures. Individually, these two countermeasures reduced the wrong-way incidence to about one-third to one-half of its original rate. This is consistent with California's Standard Sign Package, which specifies that the DO NOT ENTER and FREEWAY ENTRANCE packages be mounted with the bottom of the lower sign 600 mm (24 in) above the edge of the pavement. It also specifies that ONE WAY arrows be mounted 450 mm (18 in) above the pavement. However, the Virginia Department of Highways and Transportation (1981b) noted concern regarding the 450- mm (18-in) mounting height of the ONE WAY signs, stating that the signs may become obscured by vegetation and by guardrails (when the sign is mounted behind a guardrail). Thus, mounting height was revised for this State to 900 mm (36 in) in order to alleviate these concerns. An additional concern with lowering the mounting height of these signs is the increased potential of impacting a passenger vehicle windshield if struck by a motorist entering or exiting the freeway who strays off of the ramp and crashes into the sign support. However, wrong-way entries onto high-speed facilities can cause very serious head-on collisions in locations where there is a high incidence of wrong-way entry or a high likelihood of wrong-way entry due to geometrics. Since windshield penetration by the sign is less likely to occur at a location near the ramp terminus than at other locations because of the lower travel speeds of drivers traveling in the correct direction along the ramp (who are slowing down for a stop or a signalized turn) and of drivers making the wrong-way movement (who are accelerating from a turn), the anticipated benefit of increased sign conspicuity and prevention of wrong-way freeway entries is judged to significantly outweigh the risk of sign penetration.

California uses the DO NOT ENTER and WRONG WAY signs together on a single signpost, with the WRONG WAY sign mounted directly beneath the DO NOT ENTER sign (the Do Not Enter Package). This sign package is placed on *both* sides of the ramp. The California Standard specifies that large FREEWAY ENTRANCE signs (1200 mm x 750 mm [48 in x 30 in]) be placed on on-ramps, but the location of the sign package (FREEWAY ENTRANCE sign, plus route shield, cardinal direction sign, and down diagonal arrows) should not be controlled by the use of the larger signs. Smaller signs (900 mm x 525 mm [36 in x 21 in]) may be used for proper placement, if necessary. For off-ramp signing, the Standard specifies the use of at least one Do Not Enter Package (DO NOT ENTER and WRONG WAY signs) to be placed to fall within the area covered by the car's headlights and visible to the driver from the decision point on each likely approach; three or four packages may be required if the off-ramp is split by a traffic island. In addition, ONE WAY arrows should be placed as close to the crossing street as possible. The *MUTCD* standard sizes for the DO NOT ENTER and WRONG WAY signs are 750 mm x 750 mm (30 in x 30 in) and 900 mm x 600 mm (36 in x 24 in), respectively. California uses sizes of 900 mm x 900 mm; 1200 mm x 1200 mm; and 1800 mm x 1800 mm (36 in x 36 in; 48 in x 48 in; and 72 in x 72 in) for the DO NOT ENTER sign and 900 mm x 525 mm and 1800 mm x 525 mm (36 in x 21 in and 72 in x 21 in) for the WRONG WAY sign. As they are retrofitted and newly installed, the Do Not Enter sign packages in California have high-intensity sheeting (Copelan, 1989).

Increases in conspicuity distance have been reported in the literature on fluorescent signing. As stated earlier in Chapter I for Design Element L, Burns and Pavelka (1995) found that signs with fluorescent red sheeting were detected by 90 percent of the participants in a field study conducted at dusk. Only 23 percent of the subjects were able to detect the standard red signs under the same lighting conditions. To improve the daytime conspicuity of DO NOT ENTER and WRONG WAY signs, as well as conspicuity of these signs under low-luminance conditions (dawn and dusk), fluorescent red sheeting is recommended. In addition, use of retroreflective sheeting that provides for high brightness at the wide observation angles typical of the sign placements and distances at which these signs are viewed (e.g., 1.0+ degrees), as well as lowering the sign heights for these signs, will enhance their nighttime conspicuity under low-beam headlight illumination.

Turning to a consideration of pavement markings, Tamburri (1969) found that a white pavement arrow placed at all off-ramps pointing in the direction of the right-way movement can be effective in reducing the number of wrong-way maneuvers. However, Parsonson and Marks (1979) found that at a parclo AB loop off-ramp that has its crossroad terminal adjacent to the on-ramp, standard pavement arrows, lowered DO NOT ENTER and WRONG WAY signs, trailblazer signs, and a 600-mm-(24-in-) wide stop bar were not sufficient, as the ramp still showed a rate of 22.3 wrong-way movements per month. Large pavement arrows (7.3 m [24 ft] long) and yellow ceramic buttons (with a diameter of 200 mm [8 in]) to form a median divider on the crossroad were also required. It was specified that the ceramic buttons should touch each other to form a continuous, unbroken barrier, and should extend far enough toward the interchange structure (the freeway) to prevent a wrong-way driver from avoiding the buttons by turning early. The length required is typically 30.5 m (100 ft). The addition of the ceramic buttons reduced wrong-way maneuvers from a rate of 88.6 per month to a rate of 2.0 per month. Campbell and Middlebrooks (1988) also found that installing yellow ceramic buttons to the extension of the centerline of the crossroad to aid in channelizing left-turning traffic entering the freeway, in combination with countermeasures employed by the Georgia Department of Transportation as standard practice—trailblazer sign, 450-mm-(18-in-) wide stop line at the end of the off-ramp, 5.5-m-(18-ft-) long arrow pavement marking, and lowered WRONG WAY and DO NOT ENTER signs—reduced wrong-way maneuvers. It was also recommended in the Parsonson and Marks (1979) study that the two-piece, 7.3-m-(24-ft-) long arrow pavement marking (part of the California standard) be adopted. This use of the wrong-way arrow is described in *MUTCD* section 3B.19 and is shown in *MUTCD* figures 3B-22 and 3B-23.

The control of wrong-way movements on freeways and expressways, a related problem, may be accomplished through the use of lane control signals (LCS). The *MUTCD* (section 4J.0) defines lane control signals as "special overhead signals that permit or prohibit the use of specific lanes of a street or highway or that indicate the impending prohibition of their use." The meanings of LCS indications (steady downward green arrow, steady yellow X, and steady red X) are defined in *MUTCD* section 4J.02 An LCS provides real-time information to motorists about which downstream freeway lanes are open (green downward arrow), which are closed (red X), and which lanes are about to be closed (yellow X) either because of an incident downstream or because the lane is a reversible lane. Drivers should vacate lanes over which an LCS displays a yellow X, and they should not enter lanes designated by a red X. Safe and effective responses to these indications by older drivers hinge upon the same visual target detection and recognition processes that have been documented elsewhere in this Handbook as declining systematically with advancing age.

Ullman, Parma, Peoples, Trout, and Tallamraju (1996) conducted legibility studies of commercially available lane control signals (LCS) being used in freeway traffic management systems throughout Texas. Subjects included drivers ages 16-44 and drivers age 65 and older. In the first study, subjects were seated in the driver's seat of a test vehicle that was positioned 460 m from an overhead sign structure that displayed three fiber-optic LCS heads mounted side by side. The subjects were shown one symbol presented on one LCS for a 1.5-s duration, and were asked to describe the symbol and the color. If a subject could not correctly identify the color and symbol, he or she moved closer to the LCS until the color and symbol could be correctly identified. The study was conducted during the day. The symbols on all three LCS heads measured 356 mm (14 in) in height, but the arrangement of the pixels varied for the three sign manufacturers. Signal 1 utilized a single-stroke arrangement of pixels, arranged using 13-mm (0.5-in) lenses spaced 25 mm (1 in) apart. Signal 2 utilized a double-stroke arrangement of pixels, arranged using 15-mm (0.6-in) lenses spaced 38 mm (1.5 in) apart. Signal 3, besides displaying the three standard *MUTCD* symbols, was also able to display a yellow arrow pointing downward, or pointing to the left or right. Signal 3 utilized a double-stroke arrangement of pixels for the green arrow, the red X, and the yellow X, but utilized a single-stroke arrangement for the downward and diagonal yellow arrows. The lenses measured 4 mm (0.15 in), and spacing was 18 mm (0.7 in). Figure 21 presents the pixel layout of the three LCS heads used in this research. The legibility distances by symbol type are presented in table 35 by signal type and driver age.

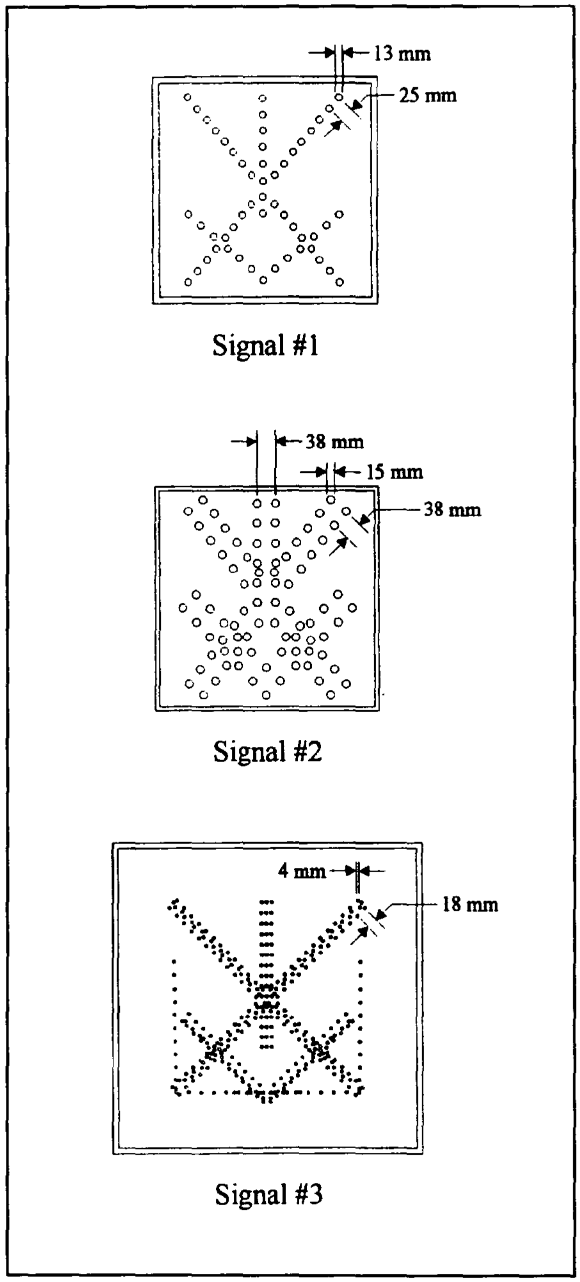

Figure 21. Pixel layout of LCS heads employed in research conducted by Ullman et al. (1996).

Results indicate that median legibility distances for the older drivers viewing the red X, yellow X, and green arrow were 91 to 198 m (298 to 649 ft) shorter than those for the younger drivers. The biggest discrepancies between younger and older drivers were for the red X on Signal 2 and the green down arrow on Signal 1. Signal 3 produced the most uniform legibility distances for all symbols. The authors state that comments from the older drivers indicated that the double-stroke pixel arrangement combined with the larger pixel lenses on Signal 2 caused irradiation effects, resulting in older drivers' inability to identify the shape of the red "glow" until they were 50 percent closer to the signal than younger drivers. The actual cause of the poor legibility of the green down arrow on Signal 1 for older drivers was unknown, but it was hypothesized by the researchers to be a result of being slightly more "blue" than the green down arrows on the other two signals.

On the other hand, although the yellow downward and diagonal arrows displayed on Signal 3 are not standard *MUTCD* symbols for an LCS, they showed the highest legibility distance of all symbols for the older drivers, and yielded a legibility distance that was 30 m shorter than the younger drivers' legibility distance. These symbols were created with a single line of closely spaced small pixels. The legibility distance was greater than that of the double-stroke yellow X presented on the same display. Results of a preliminary field study conducted in this same project indicated that the yellow X and yellow down/diagonal arrows were interchangeable, resulting in similar proportions of drivers exiting a closed lane.

The authors also studied the effects of age, dirt accumulation, and regular maintenance on LCS legibility. The combined effect of age and dirt accumulation significantly reduced the visibility of the green arrow and red X for both younger and older driver groups. The signal had been installed for 6 years and it had an 18-month accumulation of dirt since its last cleaning. Subjects viewed the symbols before and after cleaning and replacement of each of the 50-W halogen light bulbs in the signal face. The legibility distances before maintenance and cleaning ranged from 183 to 335 m (600 to 1099 ft) for drivers under age 45, and 107 to 274 m (351 to 899 ft) for drivers age 65 and older. After maintenance and cleaning, visibility distances increased significantly for both driver groups. Median visibility distances for the younger drivers ranged from 259 m to 381 m (850 to 1250 ft), and for the older drivers, they ranged from 213 to 335 m (699 to 1099 ft). There appears to be permanent loss of visibility for the red and green symbols over time; after maintenance, the legibility distances for both driver groups were two-thirds to three-fourths of those shown for the new LCS. The effect of dirt and signal age did not affect the legibility distance for the yellow X.

Table 35. Median legibility distance (meters) for lane-control signals, as a function of driver age, LCS type, and LCS symbol.
Source: Ullman, Parma, Peoples, Trout, and Tallamraju (1996).

Driver Age	Signal 1			Signal 2			Signal 3				
	Red X	Yellow X	Green Down Arrow	Red X	Yellow X	Green Down Arrow	Red X	Yellow X	Green Down Arrow	Yellow Diagonal Arrow	Yellow Down Arrow
Young/ Middle-Aged (16-44)	366	457	396	335	457	457	427	457	457	427	457
Older (65+)	274	274	198	168	350	290	305	274	335	396	427

1 m = 3.28 ft

III. ROADWAY CURVATURE AND PASSING ZONES

The following discussion presents the rationale and supporting evidence for Handbook recommendations pertaining to these four design elements (A–D):

A. Pavement Markings and Delineation on Horizontal Curves
B. Pavement Width on Horizontal Curves
C. Crest Vertical Curve Length and Advance Signing for Sight-Restricted Locations
D. Passing-Zone Length, Passing Sight Distance, and Passing/Overtaking Lanes on Two-Lane Highways

A. Design Element: Pavement Markings and Delineation on Horizontal Curves

Table 36. Cross-references of related entries for pavement markings and delineation on horizontal curves.

Applications in Standard Reference Manuals			
MUTCD (2000)		AASHTO *Green Book* (1994)	*Traffic Engineering Handbook* (1999)
Sect. 1A.13, *centerline markings, edgeline markings, object markers, raised pavement marker, retroreflectivity, & road delineators* Sects. 3A.05 & 3A.06 Sect. 3B.01, Items A & B Figs. 3B-5a & 3B-5b Table 3B-1 Sect. 3B.04	Sect. 3B.06 Sect. 3B.11 Table 3B-2 Sects. 3C.01 & 3C.03 Fig. 3D-1 Sects. 3D.01 through 3D.04 Table 3D-1 Sect. 6F.65	P. 314, Para. 7 P. 315, Paras. 1 & 3 P. 453, Sect. on *Signing and Marking* P. 534, Para. 4	P. 434, Sect. on *Edge (Fog) Lines* P. 440, Sect. on *Raised Pavement Marker*

Pavement markings and delineation devices serve important path guidance functions on horizontal curves, particularly under adverse visibility conditions, at twilight, and at nighttime. They provide a preview of roadway features ahead and give the driver information about the vehicle's lateral position on the roadway. Delineation must provide information that results in recognition of the boundaries of the traveled way, both at long preview distances (5 to 8 s of travel time) and at more immediate proximities (within 1 s of travel time) where attention is directed toward instant-to-instant vehicle control responses.

Surface pavement markings in current practice may vary along four dimensions: (1) brightness, (2) width, (3) thickness, and (4) the addition of structure to "thick" applications. Stripes of increased thickness have an advantage in wet weather because the material is more likely to protrude above the level of surface water and to provide a degree of retroreflectivity greater than that provided by thinner applications of paint. Also, the commercially available structured stripes (tapes) are brighter than other marking treatments, even under dry conditions. This is due to the ability of the raised element of the structure to reflect more light back to the driver than would a horizontal surface. Even greater benefits are provided by raised retroreflectorized treatments, including raised pavement markers (RPMs), post-mounted delineators (PMDs), and chevron signs, which may be used to improve the nighttime visibility of delineation and to indicate roadway alignment.

A number of driver visual functions that have an impact on the use of pavement markings and delineation show significant age-related decrements: dynamic acuity, contrast sensitivity, dark adaptation, and glare recovery. Dynamic visual acuity (DVA) includes the ability to resolve the details of a high-contrast target that is moving relative to an observer. Activities that rely on dynamic acuity include making lateral lane changes and locating road boundaries when negotiating a turn. In these situations, greater speeds are associated with poorer DVA. Contrast sensitivity influences the response to both sharply defined, bright versus dark visual targets, and those with grayer, less distinct edges. In general, older adults tend to have decreased contrast sensitivity (Owsley, Sekuler, and Siemsen, 1983). This loss is more pronounced at lower light levels (Sloane, Owsley, and Alvarez, 1988; Sloane, Owsley, and Jackson, 1988) and is associated with a heightened sensitivity to glare (Wolf, 1960; Fisher and Christie, 1965; Pulling, Wolf, Sturgis, Vaillancourt, and Dolliver, 1980). The findings of Blackwell and Blackwell (1971) indicate that a 60-year-old observer needs approximately 2.5 times the contrast as a 23-year-old observer for the same level of visibility.

Highway research studies that have varied one or more of the four dimensions of pavement markings are discussed below, along with studies on the effectiveness of RPMs, PMDs, chevron signs, and combinations of delineation treatments. Age differences are reported wherever data are available.

An early study of surface pavement markings, carried out with an interactive driving simulator plus field evaluations, concluded that driver performance—measured by the probability of exceeding lane limits—was optimized when the luminance (brightness) contrast between pavement markings and the roadway was 2.0 (Blackwell and Taylor, 1969). A study by Allen, O'Hanlon, and McRuer (1977) also concluded that delineation contrast should be maintained above a value of 2.0 for adequate steering performance under clear night-driving conditions. In other words, these studies have asserted that markings must appear to be at least three times as bright as the road surface, because contrast is defined as the difference between target and background luminance, divided by the background luminance alone. A difficulty with these studies, however, is that their data were not derived from—and thus are not representative of—normatively aged older drivers. The ideal viewing conditions assumed by Allen et al. (1977) also disregard the effects of glare, as well as adverse visibility, and both factors have a disproportionate impact on the performance of older drivers. In Blackwell and Taylor's work, a minimum preview time of 3 to 4 s was recommended for accurate maneuvering under adverse conditions. However, more conservative estimates of preview time to accommodate older drivers (e.g., 5 s) have frequently appeared in the literature.

Freedman, Staplin, Gilfillan, and Byrnes (1988) showed significant performance decrements for 65-year-old drivers, as compared with 35-year-old drivers, for the visibility distance of 100-mm-(4-in-) wide pavement stripes on a simulated wet roadway. Staplin, Lococo, and Sim (1990) confirmed the need for higher levels of line brightness for older drivers in a simulator study, where the contrast for a 100-mm-(4-in-) wide white edgeline was continuously varied within a 40-step range in a method of limits. Under simulated opposing headlamp glare conditions, subjects ages 65–80 required an increase in contrast of 20 to 30 percent over a younger sample to correctly discern downstream curve direction at criterion viewing distances.

To accommodate less capable older drivers, this study's results indicated that a 300-percent increase in stripe brightness versus that required by younger drivers may be warranted.

To describe the magnitude of the effects of age and visual ability on delineation detection/recognition distance and retroreflective requirements for threshold detection of pavement markings, a series of analyses using the Ford Motor Company PC DETECT computer model (see Matle and Bhise, 1984) yielded the stripe contrast requirements shown earlier in this Handbook in table 9 for Design Element F (Treatments/Delineation of Edgelines, Curbs, Medians, and Obstacles) in the Rationale and Supporting Evidence section for Intersections (At-Grade). PC DETECT is a headlamp sight-distance model that uses the Blackwell and Blackwell (1971, 1980) human contrast sensitivity formulations to calculate the distance at which various types of targets illuminated by headlamps first become visible to approaching drivers, with and without glare from opposing headlights. The top 5 percent (most capable) of 25-year-olds and bottom 5 percent (least capable) of 75-year-olds were compared in this analysis.

The more realistic operating conditions modeled as described above, together with the widely cited multiplier for older observers advocated in the seminal work by Blackwell and Blackwell (1971), support the recommendation that an in-service pavement edge-striping contrast value on horizontal curves maintained at or above 5.0 is appropriate to accommodate the needs of the large majority of older drivers on highways and arterials without median separation between opposing directions of traffic. Where a median barrier (e.g., concrete safety-shaped barrier) high enough to shield drivers from direct view of oncoming headlights is present, or where median width exceeds 15 m (49 ft), a horizontal curve edgeline contrast value of 3.75 or higher is recommended. It is important to note that these recommendations are not limited to standard striping width (100 mm [4 in]). *Where wider pavement markings are implemented, either as general or spot treatments, the <u>same</u> contrast values apply.*

This leads logically to discussion of stripe width. This is one characteristic that has been advanced as a countermeasure to accommodate older drivers, at least on roadways 6.7 m (22 ft) or more in width. The use of wider stripes has been advocated both as a general treatment and as a spot treatment on curves. A survey of State highway agencies by Wright (1983) found that engineers believe that treatments such as chevrons, delineators, and warning signs are more effective than surface markings for spot improvements at curves. Deacon (1988), who has concluded that 200-mm-(8-in-) wide edgelines should be used instead of standard 100-mm-(4-in-) wide edgelines on two-lane, rural highways, states that "... while this finding is not based on benefits to older drivers, older drivers will share—probably proportionally more—the safety benefits with others who travel these highways during periods of impaired visibility."

Relevant work in this area includes a report by Good and Baxter (1986) that 150-mm-(6-in-) wide edgelines result in more favorable driver control than 75-mm-(3-in-) wide edgelines for "short-range delineation," which was defined as that which is useful to the driver for tracking the roadway at night under poor-visibility conditions. In addition to the assumption that a wider stripe will provide greater visibility distances and be more conspicuous to older drivers, there is evidence from a study by Hughes, McGee, Hussain, and Keegal (1989) that 200-mm-(8-in-) wide edgelines offer the potential for cost-effective application. This conclusion is based on the finding that for 200-mm-(8-in-) wide edgelines to be a cost-effective replacement for 100-mm-(4-in-) wide edgelines, crashes need to be reduced by only 0.7 percent when the daily traffic exceeds 1,000 vehicles.

However, while the use of a wide edgeline is conceptually attractive for improving older driver performance, the complete operational and safety benefits are not at all clear. For example, Hall (1987) reported that wide edgelines do not reduce the incidence of run-off-road (ROR) crashes, nor do they reduce the incidence of such crashes at night or on curves. A study by Cottrell (1988) also showed that the use of wide edgelines does not reduce the risk of crashes on curves or at night; still, he agrees that the use of wide edgelines only in the vicinity of curves, while retaining conventional edgelines on tangents, could be an effective spot improvement. Lum and Hughes (1990) have expressed concern over both the Cottrell and the Hall studies because the number of miles sampled was small. And finally, in a simulator study conducted to determine the most effective horizontal curve delineation treatments to accommodate older drivers, Pietrucha, Hostetter, Staplin, and Obermeyer (1996) found that although the recognition distance for a 200-mm-(8-in-) wide white edgeline at an in-service brightness level (ISBL) in combination with a standard yellow centerline produced longer recognition distances among the older driver sample (mean recognition distance: 66.3 m [217.5 ft]) compared to a 100-mm-(4-in-) wide white edgeline at ISBL in combination with a standard yellow centerline (mean recognition distance: 54.7 m [179.4 ft]), the difference was not significant.

Thus, wider edgelines deserve consideration wherever practical, particularly as spot treatments on horizontal curves, to accommodate the difficulties older drivers have with visibility at night. And it may be inferred from these various studies of stripe width that markings that are maintained at or above the recommended contrast levels *and* are wider than the conventional 100-mm (4-in) treatment will provide the greatest benefit to older drivers. What is important to remember is that contrast remains the preeminent factor in stripe visibility, and increased width alone does not substitute for lower-than-recommended contrast levels.

A current area of investigation, which potentially could lead to the development of standards directly impacting stripe visibility, is the measurement of the retroreflectivity of pavements and pavement markings. There are currently no pavement marking retroreflectivity requirements specified in design manuals, although the *Roadway Delineation Practices Handbook* (Migletz, Fish, and Graham, 1994) states that several separate studies have concluded that the value of approximately 100 millicandelas per lux per square meter (mcd/lux/m^2) is the minimum value for the coefficient of reflected luminance (R_L) for pavement markings. More common is the expression of delineation (pavement marking) retroreflectivity in millicandelas per square meter per lux (mcd/m^2/lux), or the amount of reflected light per unit area of striping material, as a function of the incident illumination level. For present purposes, what is important is that for a given amount of incident illumination (i.e., from a vehicle's headlight beam distribution), the ratio of mcd/m^2/lux returned from the pavement marking to the mcd/m^2/lux returned from the adjacent pavement surface generally describes the brightness of the treatment as viewed by an approaching motorist.

Graham, Harrold, and King (1996) conducted a field study at night on public roadways to determine the minimum pavement marking retroreflectivity requirements to accommodate older drivers. Thirty-six drivers ages 60 to 80, with a mean age of 71.3 years, and 29 drivers ages 20 to 59 rated the adequacy of 60-m-long white and yellow stripes on tangent highway sections on clear, dry nights under low-beam headlight illumination only. Using a 1980 model four-door sedan as the test vehicle, the retroreflectance values of the markings ranged from 28 to 301 mcd/m^2/lux. In 12 of the 14 locations where the pavement markings measured 100 mcd/m^2/lux or higher, they were rated as adequate or above by at least 85 percent of the subjects age 60 and older. Markings with retroreflectivity levels of 142 mcd/m^2/lux or higher were rated as adequate or above by 95 percent or more of drivers age 60 and older. There was no significant effect of marking color, a result replicated by Schnell and Zwahlen (1996).

Graham, Harrold, and King (1996) also documented the effects of a dirty windshield and headlights on pavement marking visibility. They made measurements on a sample of in-service vehicles and found that for the 85th percentile vehicle, the light transmitted through a clean windshield was increased by 8 percent compared to an unclean windshield, and that cleaning the headlamps increased target luminance by 12 percent. Furthermore, they note that vehicles in use from the mid-1990s have headlight systems that may provide less reflected light from pavement markings than the headlight system on the 1980 test vehicle.

Next, Garvey, Gates, and Pietrucha (1997) addressed delineation as an area where engineering improvements could accommodate the needs of older drivers. Their report indicates that the very best older drivers will require 130 mcd/m^2/lux, whereas the majority of the older driver population will require 300 mcd/m^2/lux. Jacobs, Hedblom, Bradshaw, Hodson, and Austin (1995) performed a field study on the visibility of a 0.1-m- (4-in-) wide by 3.0-m- (10-ft-) long isolated centerline located 3.7 m (12 ft) from the right edge of the road, with approximately one-third of their subject sample being between the ages of 50 and 60. Results were interpreted in relation to the visibility distance needed for a 5-s preview of road heading at varying speeds. It was found that a pavement marking retroreflectivity level of 100 mcd/m^2/lux was able to provide the required visibility distance only at a speed of 24 km/h (15 mi/h). To achieve a 5-s preview distance (123 m, or roughly 400 ft) at 88 km/h (55 mi/h) by the 50th percentile driver in this study—who was under 60 years old—required a stripe retroreflectance of 1000 mcd/m^2/lux. The vehicle used in the study was a 1993 model four-door sedan.

The development of retroreflectivity requirements for pavement markings is complicated by the geometric relationships that must be taken into account—including entrance and observation angle specifications, which are sensitive to driver eye height, headlight type and height, the longitudinal separation from the target marking, and the driver/headlight lateral position in the travel lane—as well as the lack of data regarding the retroreflectivity of different types of pavement surfaces under wet and dry conditions and different degrees of wear. Perhaps the most rigorous work in this area to date has been reported by Zwahlen and Schnell (1998), who conducted studies to define the performance levels of markings required to provide a 62-year-old design driver with a preview time of 3.65 s. Finding no reliable data on the reflective properties of road surfaces under an automobile headlamp geometry (i.e., with observation angles less than 1 degree), the researchers conducted luminance and illuminance measurements for two bituminous/asphalt (worn and relatively new) and two concrete (worn and relatively new) roadway surfaces in the field. The results of the measurements indicated that the new asphalt road surface (R_L=20 mcd/m^2/lux) was substantially less reflective than the weathered, worn asphalt surface (R_L=40 mcd/m^2/lux). In comparison, the worn concrete road surface (R_L=28 mcd/m^2/lux) was considerably darker than the new concrete road surface (R_L=55 mcd/m^2/lux). Their matrix of readings for each pavement type as a function of entrance angle and observation angle were included in a subsystem of the CARVE (Computer-Aided Road-Marking Visibility Evaluator) model that was a key product of their research. Zwahlen and Schnell (1998) subsequently selected old asphalt as the road surface for a controlled field study of pavement-marking visibility. They also selected an entrance angle of 88.7 degrees and an observation angle of 1.05 degrees to represent the vehicle/observer geometry of an "average large car driven by an average size adult."

The Zwahlen and Schnell (1998) controlled field study used younger and older drivers with near normal visual performance to quantify the visibility distance of pavement markings on fully marked two-lane rural roads (white edgelines, dashed yellow centerline) to calibrate the CARVE computer model. The older driver group contained five males and five females, with an average age of 68.3 years, and the younger driver group contained five males and five females, with an average age of 23.2 years. The authors note that their use of only subjects with healthy vision suggests that the visibility distances obtained in the study are likely to be longer than what would be obtained with a larger sample that represented the U.S. driver population, including individuals with diminished visual capabilities. The study measured detection distances for 100-mm- (4-in-) wide pavement markings under low- and high-beam headlight illumination.

Based on the results of Zwahlen and Schnell (1998), the CARVE model was exercised to yield recommended retroreflectivity levels for white and yellow pavement markings under dry conditions, assuming a typical H6054-type vehicle headlamp. The 3.65-s preview time noted earlier was retained for roads with no raised treatments; when RPMs are used in addition to pavement markings, the authors revised the preview time downward to 2.0 s. The (white) edgeline minimum required retroreflectivity (R_L) values emerging from this effort are shown in table 37.

The specification of R_L values for pavement markings is likely to remain a difficult problem for some time, as indicated in the preceding discussion. It may also prove to be largely an academic issue. It is the effective contrast of a marking against the surrounding pavement area that a driver's visual system responds to, and which determines both (detection) performance and subjective comfort with the information provided by such treatments. While this index can be arrived at through complicated calculations based on the retroreflectivity levels of pavements and markings, it can also be done through a simple, direct measurement of stripe and pavement luminance under the observation conditions of interest (e.g., nighttime, low-beam headlights, dry pavement) at any site where the need for restriping must be determined. Based on luminance meter readings of stripe luminance and pavement luminance obtained at any practical distance, the dimensionless number denoting contrast level for the pavement marking can be calculated as follows:

$$Luminance\ contrast\ (C) = \frac{Luminance_{stripe} - Luminance_{pavement}}{Luminance_{pavement}}$$

What deserves emphasis—aside from the fact that when one measures contrast, one is measuring what drivers actually see—is just how straightforward it is to obtain this information. An engineer or technician, with a single hand-held piece of equipment (luminance meter) can quickly obtain the measures needed for the contrast calculation under any operating condition of interest.

Table 37. Minimum required R_L (mcd/m^2/lux) recommended by Zwahlen and Schnell (1998) for roads consisting of two white edgelines and a dashed yellow or white centerline.

Vehicle Speed (km/h)	Vehicle Speed (mi/h)	Minimum Required R_L (mcd/m^2/lux) for White Edgeline	
		Without RPMs (Preview Time = 3.65 s)	With RPMs (Preview Time = 2.0 s)
0-40	0-25	30	30
41-56	26-35	50	30
57-72	36-45	85	30
73-88	46-55	170	35
89-104	56-65	340	50
105-120	66-75	620	70

This discussion now turns to raised and reflectorized treatments. Raised pavement markers have received widespread use because they provide better long-range delineation than conventional pavement markings, particularly under wet conditions. When used on a road edge, they also provide brighter peripheral cues, which could be advantageous to the older driver for path guidance. Over time, however, RPMs also are subject to loss of their initial retroreflectivity; in colder climates, RPMs may be damaged by plowing operations.

Deacon (1988), in his review of research on delineation and marking treatments that he believed would be of particular benefit to the older driver, found that highways with RPM-enhanced centerlines had lower crash rates than those with painted centerlines only. The average reduction in crash rates was approximately (0.5 crashes per million vehicle-miles). Zador, Stein, Wright, and Hall (1986) observed that after-modification vehicle paths were shifted away from the centerline on right and left curves with RPMs mounted on both sides of the double yellow centerlines, and that placement changes were largest with RPMs compared to PMDs and chevrons. It has also been observed that RPMs placed in the centerlines and edgelines at pavement width reductions at narrow bridges produce significant reductions in 85th percentile speeds and centerline encroachments (Niessner, 1984). On two-lane rural curves, RPMs, in conjunction with the double yellow centerline, have been recommended.

An RPM spacing study was conducted by Blaauw (1985), who tested several RPM patterns on 200-m (656-ft) radius and 1000-m (3281-ft) radius horizontal curves using a visual occlusion technique. White RPMs were used for the tests at spacing distances of approximately 12.2 m, 24.4 m, and 36.6 m (40 ft, 80 ft, and 120 ft). On 200-m (656-ft) radius curves, the 24.4-m and 36.6-m (80-ft and 120-ft) spacings led to speed reductions and lane errors. Based on these results, it was recommended that on curves of this severity, the spacing of RPMs be restricted to 12.2-m (40-ft) spacings. In general, no differences between treatments were observed for the more gentle, 1000-m (3281-ft) radius curves.

Accordingly, this Handbook recommends RPM installation at standard (12.2-m [40-ft]) spacings on all horizontal curves with radii below 1000 m (3280 ft).

Roadside delineators and treatment combinations are also important to this discussion. Because of its increasing use throughout the United States, and because it accommodates different types of sheeting in varying amounts and different designs, the primary roadside delineation device of current interest is the flat, flexible post. The general crash data have shown that the installation of PMDs is associated with lower crash rates for highway sections with or without edgelines (Bali, Potts, Fee, Taylor, and Glennon, 1978; Schwab and Capelle, 1979). Deacon (1988) confirmed that installation of PMDs lowered crash rates for sections with or without edgelines. The reduction in crash rates resulting from the installation of these delineators averaged (1.0 crashes per million vehicle-miles). Thus, especially for lower functional classification roadways where the use of enhanced (e.g., wider) edgelines may be limited (due to pavement width restrictions), existing data suggest that PMDs can be an effective countermeasure.

In a driver performance study evaluating the effects of chevron signs, PMDs, and RPMs, both Johnston (1983) and Jennings (1984) found that driver performance on sharp curves was most favorable when chevrons were used. With chevrons, drivers followed a better path around the curve (defined in terms of the ratio of the vehicle's instantaneous radius to the actual curve radius). These studies also revealed that drivers use a corner-cutting strategy, and that chevron signs and PMDs facilitated this strategy. On right curves with chevrons, drivers had an average mid-curve placement closest to the centerline. On left curves with chevrons, vehicle placement was not significantly different. In the Good and Baxter (1986) study, chevron signs had a detrimental effect on control behavior, but were rated favorably by drivers in reducing task difficulty. Zador et al. (1986) found that chevrons (as well as RPMs) tend to shift vehicles away from the centerline on right and left curves, while PMDs shift vehicles away from the centerline on right curves. A particular advantage of chevrons with high-intensity retroreflective sheeting was demonstrated for drivers age 65 and older in a study by Pietrucha, Hostetter, Staplin, and Obermeyer (1996), when used in combination with other treatments.

The Pietrucha et al. (1996) study was specifically directed toward the difficulties older drivers have with horizontal curve delineation elements and the possible benefits of brighter materials, larger target sizes, redundant and/or multi-dimensional cues using combinations of elements, and novel designs or configurations of elements. Twenty-five distinct delineation/pavement marking treatments (a baseline treatment and 24 enhancements) were initially presented to subjects in 3 driver age groups (18–45, 65–74, and 75 and older). The baseline treatment was a 100-mm-(4-in-) wide yellow centerline at in-service brightness level (ISBL). The 24 treatments varied according to the presence/absence of an edgeline, edgeline width, whether the edgeline was enhanced with RPMs, whether the centerline was enhanced with RPMs, and the presence/absence of off-road elements and their characteristics (material, color, brightness, and/or spacing). Measures of effectiveness were downstream roadway feature recognition (subjects were required to report the direction in which the roadway curved) and recognition distance in a 35-mm simulation of nighttime driving. Treatments that included the addition of RPMs to *both* the centerline and edgeline and all treatments that included delineating the roadway edge with high-intensity chevrons or high-intensity PMDs resulted in significantly longer mean recognition distances when compared with the baseline treatment, across all age groups. For the subjects age 65 and older, only a subset of the treatments with delineated roadway edges resulted in significantly longer mean recognition distances, due to the increased variance among older subjects' data. Next, field evaluations were conducted with a subset of the most promising treatments. The treatment with the longest recognition distance for both age groups consisted of a 100-mm- (4-in-) wide yellow centerline at ISBL with yellow RPMs at ISBL and standard spacing, a 100-mm- (4-in-) wide white edgeline, and fully reflectorized T-post delineators with standard spacing. For the 152.4-m (500-ft) radius of curvature used in this study, spacing for the PMDs was 19.8 m (65 ft). This treatment included PMDs that were fully retroreflectorized (i.e., retroreflective material extended from the top of the post to the ground and provided more retroreflective area than the standard posts most frequently used).

Blaauw (1985) tested combinations of PMDs and RPMs, resulting in the following recommendations: (1) RPMs exclusively at the center are favorable for lateral vehicle control inside the lane (short-range delineation), but are less adequate for preview information on the lane to be followed (long-range delineation); therefore, it is necessary to delineate both lane boundaries; (2) effective centerline delineation can be realized with RPMs; (3) delineation at the outside of the traffic lane can be realized with RPMs at the location of the lane boundary or with PMDs spaced laterally at 1.5 m (5 ft)—both configurations are equally efficient, but PMDs at an approximate 3.7-m (12-ft) spacing are less efficient; and (4) RPMs at the location of the center and/or lane boundaries must be applied with a maximum spacing distance of 12 m (40 ft) on a curve with a 200-m (656-ft) radius or less.

In a laboratory study of drivers' responses to videotapes of four rural horizontal curves, six levels of delineation were studied by Nemeth, Rockwell, and Smith (1985). Seventy-eight drivers ages 18 to 63 participated. The levels of delineation included: no delineation; centerline only; centerline plus edgeline; centerline plus edgeline plus PMDs; centerline plus edgeline plus RPMs; and centerline plus edgeline plus PMDs plus RPMs. Subjects were required to identify precisely the instant that they could detect the presence of a curve (left or right) and then express their level of confidence with their response. The largest increase in detection distance was associated with the addition of RPMs and PMDs to the centerline and edgeline treatments, respectively. While the treatment condition that included all delineators (PMDs and RPMs, in addition to the centerline and edgeline) produced the greatest detection distance (an increase in 148 percent over centerline and edgeline delineation only), the most significant change occurred with the addition of the RPMs to a road that initially had only the centerline and edgeline (a 112-percent increase in detection distance over centerline and edgeline only). The addition of PMDs to the centerline-and edgeline-only condition resulted in an increase in detection distance of 58 percent.

While no *specific* roadside treatment on horizontal curves is advocated in this Handbook, a recommendation for roadside delineation devices at minimum spacings keyed to curve radius appears justified by the findings reported above. Using current practice as a guide, a spacing of 12 m (40 ft) represents an average value in table 3D-1 of the *MUTCD*, Suggested Spacing for Highway Delineators on Horizontal Curves, for curves with radii from 15 to 150 m (50 to 500 ft). This value is also consistent with the 12-m (40-ft) spacing requirement for RPMs on curves with radii ≤ 200 m (656 ft) noted above.

Finally, as noted by Puvanachandran (1995), vehicle speed upon curve entry is a function of vehicle speed at the approach to the curve, and is not necessarily related to the sharpness of the curve. This underscores the importance of efforts to reduce traffic speeds on the tangent sections preceding the points of curvature. In an Insurance Institute for Highway Safety (IIHS)-sponsored study, Retting and Farmer (1998) evaluated the effectiveness of an experimental pavement-marking message in reducing excessive traffic speeds at rural and suburban two-lane roadway locations with sharp horizontal curvatures. The experimental text/symbol message employed in this study consisted of the word "SLOW" in 2.4-m- (8-ft-) high white retroreflective letters, a 2.4-m- (8-ft-) long white retroreflective left curve arrow, and a 450-mm- (18-in-) wide white retroreflective line perpendicular to the centerline of the road at both the beginning and end of the message. This message was placed 67 m (220 ft) in advance of a 90-degree curve. Traffic speeds were measured at this site on the tangent section at two points: 198 m (650 ft) and 27 m (90 ft) prior to the point of curvature, both before and after application of the markings. Speeds were also measured at a control site located on the same roadway in the opposite direction of travel, which contained a 45-degree left curve. Data were collected on Saturdays beginning at 10:30 a.m., and ending Sundays at 3:00 a.m., allowing for analysis of daytime (10:30 a.m. to 5:00 p.m.), evening (5:00 p.m. to midnight), and late-night (midnight to 3:00 a.m.) traffic speeds. All measurement periods were free from precipitation. Data were analyzed for approximately 800 passenger cars at each location; driver age was not a variable in this study.

The experimental message was associated with a significant decrease in mean speed under daytime, evening, and late-night conditions, and a significant decrease in the percentage of drivers exceeding 64 km/h (40 mi/h) under the daytime and late-night conditions, compared to the upstream and control sites. During the day, the mean speeds at the experimental site dropped from 55.2 to 53.4 km/h (34.3 to 33.2 mi/h), while the speeds at the upstream site increased from 64.7 to 67.1 km/h (40.2 to 41.7 mi/h), and the speeds at the control site increased from 63.7 to 66.1 km/h (39.6 mi/h to 41.1 mi/h). The percentage of drivers exceeding 64 km/h (40 mi/h) decreased from 9.1 percent to 3.5 percent at the experimental site, but increased from 54 to 66 percent at the upstream site, and from 47 to 62 percent at the control site. During the late-night period, the mean traffic speed decreased from 56.5 to 51.0 km/h (35.1 to 31.7 mi/h) at the experimental site, and the percentage of drivers exceeding 64 km/h (40 mi/h) decreased from 18.5

to 1.6 percent. This 10-percent decrease in speed was significantly different from the 3-percent decrease in speed measured at the upstream site. Retting and Farmer (1998) state that these average traffic speed reductions and the reductions in the proportion of high traffic speeds associated with the experimental markings are highly significant. Given the exponential relationship between fatality risk and change in velocity during collisions, even seemingly small reductions in mean traffic speeds are likely to result in significant safety benefits. The benefits of this experimental treatment in enhancing the safety of older drivers is unknown at this time and, therefore, no recommendation has been included in this Handbook. However, based on the factors that contribute to run-off-road, head-on, and rollover collisions on curves (e.g., driver impairment, fatigue, inattention, visual deficits, and excessive vehicle speed), it may be expected that such advance information would disproportionately benefit older drivers with age-related diminished visual and attentional capabilities.

B. Design Element: Pavement Width on Horizontal Curves

Table 38. Cross-references of related entries for pavement width on horizontal curves.

Applications in Standard Reference Manuals	
AASHTO **_Green Book_ (1994)**	***Traffic Engineering*** ***Handbook* (1999)**
Pp. 212-218, Sects. on *Traveled Way Widening on Curves, Derivation of Design Values, Design Values, & Attainment of Widening on Curves*	P. 366, Sect. on *Driver Behavior on Curves* P. 378, Para. 5 & Table 11-16

Roadway alignment is a key factor in unsafe vehicular operation (i.e., increasing degrees of curvature cause more crashes [Haywood, 1980]). The widening of lanes through horizontal curves, minimizing the use of controlling or maximum curvature for a given design speed, and the use of special transition curves for higher speed and sharper curve designs have all been suggested as countermeasures. Whereas in the past, lane widening has been advocated to accommodate the tracking of large trucks through curves, the present focus is on the accommodation of older drivers, whose diminished physical and perceptual abilities make curve negotiation more difficult. Lane widths on horizontal curves range from 2.7 m to 4 m (9 ft to 13 ft), but are usually 3.4 m or 3.7 m (11 ft or 12 ft) wide. Neuman (1992) recommended that when less than 3.7-m-(12-ft-) wide lanes are used, consideration should be given to widening the lane to this dimension through horizontal curves; and a further increase in width of 0.3–0.6 m (1–2 ft) may be advised to provide for an additional margin of safety through the curve for heavy vehicles. This margin of safety could also be justified in terms of its benefit to older drivers with diminished physical abilities.

Older drivers, as a result of age-related declines in motor ability, have been found to be deficient in coordinating the control movements involved in lane-keeping, maintaining speed, and handling curves (Brainin, Bloom, Breedlove, and Edwards, 1977). McKnight and Stewart (1990) also reported that older drivers have difficulty in lane-keeping, which results in frequently exceeding lane boundaries, particularly on curves. Drivers who lack the required strength, including older drivers and physically limited drivers, often swing too wide in order to lengthen the turning radius and minimize rotation of the steering wheel.

Joint flexibility is an essential component of driving skill. Osteoarthritis, the most common musculoskeletal disability among older individuals, affects more than 50 percent of the population age 65 and older (Roberts and Roberts, 1993). If upper-extremity range of motion is impaired in the older driver, mobility and coordination are seriously weakened. Older drivers with some upper-extremity dysfunction may not be able to steer effectively with both hands gripping the steering wheel rim. In a study of 83 people with arthritis, 7 percent used only the right hand to steer and 10 percent used only the left hand (Cornwell, 1987).

The general relationship between pavement width and safe driving operations has been well documented. Choueiri and Lamm (1987) reported the results of several early studies that found an association between decreasing crash frequency and increasing pavement widths. Krebs and

Kloeckner (1977) reported that for every 1-m (3.3-ft) increase in pavement width, a decrease of 0.25 in the crash rate (per million vehicle-kilometers) could be expected. Hall, Burton, Coppage, and Dickinson (1976) examined the nature of single-vehicle crashes involving fixed objects along the roadside of non-freeway facilities. They found that the majority of these types of crashes were reported as non-intersection related and occurred most frequently on weekends, at night, under adverse pavement and weather conditions, and on horizontal curves (especially outside of the curve). These crash types have high injury severity to drivers and passengers. Wright and Robertson (1979) reported that 40 and 31 percent of all fatal crashes in Pennsylvania and Maryland, respectively, were the result of a vehicle hitting a fixed object such as a tree, utility pole, or bridge abutment. In a study that focused on 600 crash sites (and 600 comparison sites) involving fixed objects, crash locations were best discriminated from comparison locations by a combination of curvature greater than 9 degrees (radius: 194 m [637 ft]) and a downhill gradient steeper than 3 percent; and, for the fatal fixed-object crash population, the crash locations were best discriminated from comparison locations by a combination of a curvature greater than 6 degrees (radius: 291 m [955 ft]) and a downhill gradient steeper than 2 percent.

Glennon and Weaver (1971) evaluated the adequacy of geometric design standards for highway curves by filming vehicles entering unspiraled highway curves with a curvature ranging from 2 to 7 degrees (radius: 873 to 249 m [2865 to 819 ft]). While driver age was not analyzed, the results of the study indicated that most vehicle paths, regardless of speed, exceed the degree of highway curvature at some point on the curve. Glennon, Neuman, and Leisch (1985) measured vehicle speed and lateral placement on horizontal curves and found that drivers tend to overshoot the curve radius, producing minimum vehicle path radii sharper than the highway curve, and that the tendency to overshoot is independent of speed. They observed that the tangent alignment immediately in advance of the curve is the critical region of operation, because at about 61 m (200 ft) before the beginning points of the curve (or approximately 3 s of driving time), drivers begin to adjust both their speed and path. Such adjustments are particularly large on sharper curves. Thus, the margin of safety in current AASHTO design policy is much lower than anticipated.

Zegeer, Stewart, Reinfurt, Council, Neuman, Hamilton, Miller, and Hunter (1990) conducted a study to determine the horizontal curve features that affect crash experience on two-lane rural roads and to evaluate geometric improvements for safety upgrading. An analysis of 104 fatal and 104 non-fatal crashes on rural curves in North Carolina showed that in more of the fatal crashes, the first maneuver was toward the outside of the curve (77 percent of the fatal crashes versus 64 percent of the non-fatal crashes). For approximately 28 percent of the fatal crashes (versus 8.8 percent of the non-fatal crashes), the vehicle ran off the road to the right and then returned to be involved in a crash. Furthermore, an analysis of 10,900 horizontal curves in Washington State, with corresponding crash, geometric, traffic, and roadway data variables, showed that the percentages of severe non-fatal injuries and fatalities were greater on curves than on tangents with the same width, where total road width (lanes plus shoulders) was ≤ 9 m (30 ft).

Zegeer et al. (1990) concluded that widening lanes or shoulders on curves can reduce curve crashes by as much as 33 percent. Specifically, table 39 shows the predicted percent reduction in crashes that would be expected on horizontal curves by widening the lanes and by widening paved and unpaved shoulders (Zegeer et al., 1990).

Table 39. Percent reduction in crashes on horizontal curves with 2.4-m (8-ft) beginning lane width as a result of widening the lane, paved shoulder, and unpaved shoulder. Source: Zegeer et al. (1990).

Total Amount of Lane or Shoulder Widening (ft)		Percent Crash Reduction		
Total	Per Side	Lane Widening*	Paved-Shoulder Widening	Unpaved-Shoulder Widening
2	1	5	4	3
4	2	12	8	7
6	3	17	12	10
8	4	21	15	13
10	5	*	19	16
12	6	*	21	18
14	7	*	25	21
16	8	*	28	24
18	9	*	31	26
20	10	*	33	29

1 ft = 0.305 m

* Values of lane widening correspond to a maximum widening of 8 ft (2.4 m) to 12 ft (3.7 m) for a total of 4 ft (1.2 m) per lane, or a total of 8 ft (2.4 m) of widening.

The evidence cited above from the engineering studies describing curve negotiation, pavement width, and crash reduction, together with the documented difficulties in lane-keeping and diminished motor abilities of older drivers, support the recommendation for a minimum pavement width (including shoulder) of 5.5 m (18 ft) on arterial horizontal curves greater than 3 degrees of curvature (radius: 582 m [1910 ft]) (see Cirillo and Council, 1986). It is understood that limited-access highways already exceed this recommended lane-plus-shoulder width. However, older drivers often report a preference for traveling on two-lane arterials, and these facilities may be deficient in this regard, especially in rural settings.

C. Design Element: Crest Vertical Curve Length and Advance Signing for Sight-Restricted Locations

Table 40. Cross-references of related entries for crest vertical curve length and advance signing for sight-restricted locations.

Applications in Standard Reference Manuals			
MUTCD (2000)	AASHTO *Green Book* (1994)		*Traffic Engineering Handbook* (1999)
Tables 2C-1 through 2C-5 Sects. 2B.24, 2C.05, 2C.26, 2C.32, & 2C.41 Sect. 5C.04	P. 46, Paras. 1 & 2 Pp. 117-127, Portion of Sect. on *Sight Distance* P. 137, Sects. on *Stopping Sight Distance Object & Passing Sight Distance Object* Pp. 279-292, Portion of Sect. on *Vertical Curves* P. 314, Para. 2	Pp. 461-462, Sect. on *Sight Distance* P. 485, Sects. on *Sight Distance & Alinement* P. 499, Para. 5 P. 722, Table IX-11 P. 723, Sect. on *Vertical Control &* Fig. IX-44 on p. 724	P. 370, Paras. 5-6 Pp. 373-374, Sect. on *Stopping Sight Distance (SSD)*

From a human factors perspective, the accommodation of older driver needs should be a high priority at sight-restricted locations because of the potential for violation of expectancy, even though the actual percentage of crashes occurring under conditions of limited (vertical) sight distance is quite small (Pline, 1996). Older drivers, as a result of their length of experience, develop strong expectations about where and when they will encounter roadway hazards and high-demand situations with increased potential for conflict. At the same time, older driver reaction time is slower in response to unexpected information, and older drivers are slower to override an initial incorrect response with the correct response. Furthermore, aging is associated with physical changes that may interfere with rapid vehicle control when an emergency maneuver is required.

Of greatest importance during the approach to sight-restricted locations are the cognitive components of driving, most notably selective attention and response speed (complex reaction time). Selective attention refers to the ability to identify and allocate attention appropriately to the most relevant targets at any given time (Plude and Hoyer, 1985). One important finding in the selective attention literature, as noted above, is that older adults respond much more slowly to stimuli that are unexpected (Hoyer and Familant, 1987), suggesting that older adults might be particularly disadvantaged when an unexpected hazard appears in the road ahead. In fact, Stansifer and Castellan (1977) suggested that hazard recognition errors can be interpreted more as attention failures than as sensory deficiencies. The selective-attention literature suggests that for adults of all ages, but particularly for older people, the most relevant information should be signaled in a dramatic manner to ensure that it receives a high priority for processing.

Next, appropriate vehicle control behaviors when unexpected hazards are encountered depend on "speeded responding," or how quickly an individual is able to respond to a relevant target once it is identified. A timely braking response when one recognizes that the car ahead is stopped or that a red signal or stop sign is present can determine whether or not there is a crash. Thus, "reaction time" or the ability to respond quickly to a stimulus is a critical aspect of successful driving. Mihal and Barrett (1976) measured simple, choice, and complex reaction times and reported that simple and choice reaction times were not correlated with crashes; however, complex reaction time was. Moreover, when only older adults were examined, the correlation with crash involvement increased from 0.27 for complex reactions for the total sample to 0.52, suggesting that the relationship was particularly significant for older adults. There is nearly uniform agreement among researchers that reaction time increases with age. In particular, studies have demonstrated a significant and disproportionate slowing of response for older adults versus young and middle-aged adults as uncertainty level increased for response preparation tasks. Preparatory intervals and length of precue viewing times appear to be crucial determinants of age-related differences in movement preparation and planning (Eisdorfer, 1975; Stelmach, Goggin, and Garcia-Colera, 1987; Goggin, Stelmach, and Amrhein, 1989).

In summary, the age-related deficits in reaction time and various aspects of attention are not independent of one another, and more than one of these mechanisms is likely to reduce driving efficiency in the older adult. Because of these deficits, sight-restricted locations pose a particular risk to older drivers, presenting a need for recommendations addressing both geometry and signing that can be reconciled with available highway research findings in this area.

Unfortunately, there is a lack of conclusive data on this subject. Kostyniuk and Cleveland (1986) analyzed the crash histories of 10 matched pairs of sites on two-lane rural roadways. The 10 limited sight distance (vertical curve) locations were defined as those below the minimum stopping sight distance (SSD) recommended by AASHTO in 1965, and ranged from 36 m to 94 m (118 ft to 308 ft). The control-site locations were defined as those that more than met the standard (SSD greater than 213 m [700 ft]). Seven of the limited sight distance sites had more crashes than the matched control sites, two were approximately equal, and one had fewer crashes (Pline, 1996). Overall, the set of sites with less than minimum SSD had over 50 percent more crashes in the study period than did the control sites.

Farber (1982) performed sensitivity analyses of the effects of change in eye height, object height, friction, and speed on SSD on crest vertical curves. He found that SSD was relatively insensitive to a reasonable range of changes in driver eye height, but was very sensitive to speed, friction, and reaction time. Thus, stopping distance on vertical curves that are of inadequate length or are substandard according to other design criteria, and where major redesign, repaving, or excavation is not feasible, could most efficiently be made safer by modifying a driver's approach speed and/or reaction time. For 88-km/h (55-mi/h) traffic, stopping distance increases 24.7 m (81 ft) for every 1-s increase in reaction time. Similarly, stopping distance decreases about 4.9 m for each 1-km/h reduction in speed (or 26 ft for each 1-mi/h reduction). A need for more effective traffic control countermeasures is thus highlighted.

A reevaluation of crest vertical curve length requirements was performed by Khasnabis and Tad (1983). These researchers reviewed the historical changes in parameters that affect the computation of SSD and evaluated the effect of these changes on the length requirements of crest vertical curves. The principal conclusions were that further tests on reaction time are needed, since the current 2.5-s reaction time may not accurately reflect the changing age distribution and composition of the driving population. In addition, the validity of the assumption of a speed differential for wet pavement conditions between design speed and top driving speed is questionable, since there is very little evidence to substantiate the assumption that all motorists are likely to reduce their speed on wet pavements. Of particular interest, Khasnabis and Tad (1983) noted that the object height of 150 mm (6 in) appears to be somewhat arbitrary (i.e., the current AASHTO design criterion), and they stated that reducing the object height to 75 mm (3 in) would improve the safety elements of crest curves.

In contrast, there are strong proponents of the position that the obstacle height criterion for design of vertical curves should be *raised* to 450 mm (18 in), or the approximate height of a passenger vehicle's rear taillights (see Neuman, 1989). Fambro, Fitzpatrick, and Koppa (1997) evaluated AASHTO's SSD model during the conduct of NCHRP project 4-42 and recommended that the object height be raised to 600 mm (24 in) in future revisions to the *Green Book*. The rationale provided by Fambro et al. (1997) includes the following points:

- Crashes involving small objects are extremely rare events and almost never result in injuries to vehicle occupants.
- Small objects are beyond most drivers' visual capabilities at the SSDs required for most rural highways (e.g., 131 m for a 90-km/h roadway), especially at night.
- Large animals (e.g., cattle, deer) and other vehicles are more realistic and more frequent hazards to drivers, and from a potential hazard standpoint, the critical object for SSD should be the smallest visible object during the day and at night that represents a hazard to the driver.
- Approximately 95 percent of the taillight heights and 90 percent of the headlight heights exceed 600 mm and, therefore, 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 these locations, a shorter object is appropriate.

While McGee (1995) has reported that available data are insufficient to definitively establish the relationship between (limitations in) vertical alignment and highway safety, and on the surface it sounds reasonable to use a height criterion corresponding to the most commonly encountered obstacle on the road (i.e., another vehicle), this approach disproportionately penalizes older drivers in those rare circumstances when a hazard (of any type) appears unexpectedly due to sight-restricting geometry. Also, the simple argument that a conclusive relationship cannot be demonstrated as justification for changing current practice is somewhat disingenuous—a significant relationship between visual acuity and crash involvement has proven to be elusive over decades of study, yet there is widespread acknowledgment that good vision is necessary for safe driving.

In consideration of potential countermeasures, since stopping distance is sensitive to decreases in speed and reaction time, any traffic control device that lowers either parameter is beneficial. In one study, a LIMITED SIGHT DISTANCE sign (W14-4 in the 1988 *MUTCD*) with a speed advisory was found to be understood by only 17 percent of the 631 respondents who passed through the study site (Christian, Barnack, and Karoly, 1981). Part of the problem may be that unlike the hazards cited by other warning signs, the phrase "limited sight distance" has no tangible manifestation, and even when drivers have topped the crest of a vertical curve, they may not be aware of the extent to which their sight distance was reduced. Freedman, Staplin, Decina, and Farber (1984) developed and tested the effectiveness of both word and symbol alternative warning devices for use on crest vertical curves using drivers ages 16 to 75. The existing LIMITED SIGHT DISTANCE sign, with or without a supplementary speed advisory panel, did not produce desirable driver responses (braking or slowing) as frequently, nor was it recalled, comprehended, recognized, or preferred as often as the alternatively worded SLOW/HILL BLOCKS VIEW sign, or an alternative symbol sign that depicted two vehicles approaching from opposite sides of a hill.

The *MUTCD* 2000 states that the HILL sign (W7-1a) is intended for use in advance of a downgrade where the length, percent of grade, horizontal curvature, or other physical features require special precautions on the part of road users. While the LIMITED SIGHT DISTANCE sign is not included in the *MUTCD* 2000, practitioners are aware of the need to alert drivers of sight restrictions due to vertical curvature and continue to use the LIMITED SIGHT DISTANCE sign in many jurisdictions. Since the SLOW/HILL BLOCKS VIEW sign is explicit in telling drivers what they should do and why they should do it, its use is recommended over both the LIMITED SIGHT DISTANCE sign and the HILL sign.

Next, several studies have shown that the use of active sign elements, such as flashing warning lights for SLOW WHEN FLASHING and MAX SPEED ___ MPH messages supplementing various standard warning signs, increases the conspicuity of the signs and results in greater speed reductions (Zegeer, 1975; Hanscom, 1976; Lanman, Lum, and Lyles, 1979; Lyles, 1981), as well as a 60-to 70-percent reduction of crashes at grade crossings compared with the static sign alone conditions (Hopkins and Holmstrom, 1976; Hopkins and White, 1977). According to Pline (1996), several agencies have experienced success with the use of flasher-augmented warning signs with the legend PREPARE TO STOP when there is limited sight distance to a signalized intersection, activated at the time of signal change (red phase).

Lyles (1980) compared the effects of warning signs at horizontal and crest vertical curves with limited sight distance (less than 152.4 m [500 ft]). Five warning devices were evaluated: (1) the standard intersection crossroad warning symbol sign; (2) a warning sign with the message VEHICLES ENTERING; (3) a sequence of two warning signs and a regulatory sign (REDUCED SPEED AHEAD, crossroad symbol, and 35 mph speed limit sign); (4) a VEHICLES ENTERING sign with constantly flashing warning lights; and (5) the same as (4), but with a WHEN FLASHING plate, with flashing warning lights activated only in the presence of crossroad traffic. Overall, the standard crossroads and VEHICLES ENTERING signs had less speed-reducing effect (0.8–3.2 km/h [0.5–2 mi/h]) than the warning/warning/regulatory sequence and the signs with warning lights (6.4–8 km/h [4–5 mi/h]). This trend was the same for both horizontal and vertical curves, and there was no significant difference between the warning/warning/regulatory sequence and the signs with warning lights. Motorists were twice as likely to recall the warning/warning/regulatory sequence and signs with warning lights than the standard signs, and a van positioned at the crossroad was also reported to have been seen more often with these sign configurations.

If the combination of advance warning messages—in particular, PREPARE TO STOP—and flashers appears to offer the greatest benefit, the activation of the flashers on the red phase only may be problematic. Drivers may associate the absence of flashers to signify an "all clear" condition, when a queue remains from the prior red phase. On the other hand, continuously activated flashers would provide no information about the status of the upcoming signal or traffic conditions (queue) resulting from a red signal, and could therefore breed disregard for the sign because it carries false information. Therefore, the present recommendation is to augment the sign panel with advance warning flashers (AWF) that are activated by the traffic signal controller prior to the onset of the yellow phase and continue to flash until the signal turns green, long enough for the expected queue to dissipate.

As reviewed above, studies have shown that, in general, approach speeds to crest vertical curves make a safe response by older drivers to a revealed obstacle unlikely given current design criteria. There is ample evidence of a significant age-related decline in capability in responding to unexpected hazards. Analyses of curve length and sight-distance requirements conclude that safety benefits will result from a lower object height criterion, while crash data analyses have prompted a move toward a higher criterion. The most prudent practice to preserve existing levels of safety as a steadily increasing segment of the driving population experiences diminished capability in responding to unexpected hazards is to retain the 150-mm (6-in) criterion. In addition, conspicuous and comprehensible warning devices should be especially beneficial to older drivers in sight-restricted situations.

D. Design Element: Passing-Zone Length, Passing Sight Distance, and Passing/Overtaking Lanes on Two-Lane Highways

Table 41. Cross-references of related entries for passing-zone length, passing sight distance, and passing/overtaking lanes on two-lane highways.

Applications in Standard Reference Manuals		
MUTCD (2000)	AASHTO *Green Book* (1994)	*Traffic Engineering Handbook* (1999)
Table 2B-1 Sects. 2B.24 & 2B.25 Tables 2C-1 through 2C-5 Sects. 2C.06 & 2C.32 Sect. 3B.02	P. 44, Para. 3 Pp. 128-136, portion of sect. on *Passing Sight Distance for Two-Lane Highways* & all of sect. on *Frequency and Length of Passing Sections* P. 223, Sect. on *Passing Sight Distance* Pp. 262-268, Sects. on *Passing Lane Sections on Two-Lane Roads, Three-Lane Sections, Turnouts, Shoulder Driving, & Shoulder Use Sections* Pp. 286-288, Sect. on *Design Controls Passing Sight Distance* Pp. 446-447, Sect. on *Passing Sight Distance* Pp. 461-462, Sect. on *Sight Distance* P. 485, Sects. on *Sight Distance* & Alinement Pp. 490-491, Sect. on *Provision for Passing* P. 500, Sect. on *Climbing Lanes on Multilane Arterials*	P. 224, Sect. on *No-Passing Zones* P. 374, Sect. on *Passing Sight Distance (PSD)* P. 399, Sect. on *Special Features* P. 427, Para. 3 & Table 12-7

The safety and effectiveness of passing zones depend on the specific geometric characteristics of the highway section, as well as on how drivers receive and process information provided by signs and pavement markings, integrate speed and distance information for opposing vehicles, and control their vehicles (braking and acceleration) during passing maneuvers. As the number of older drivers in the population increases dramatically over the years 1995–2025, many situations are expected to arise where not only the slower moving vehicle, but also the passing vehicle, are driven by an older person.

The capabilities and behavior of older drivers, in fact, vary with respect to younger drivers in several ways critical to this discussion. Studies have shown that while driving speed decreases with driver age, the sizes of acceptable headways and gaps tend to increase with age. While motivational factors (e.g., sensation-seeking, risk-taking) have been shown to play a major role in influencing the higher speeds and shorter headways accepted by young drivers, they seem to play a less important role in older driver behavior. Instead, the relatively slower speeds and longer headways and gaps accepted by older drivers have been attributed to their compensating for decrements in cognitive and sensory abilities (Case, Hulbert, and Beers, 1970; Planek and Overend, 1973).

The ability to judge gaps when passing in an oncoming lane is especially important. For some older drivers, the ability to judge gaps in relation to vehicle speed and distance is diminished (McKnight and Stewart, 1990). Depth perception (i.e., the ability to judge the distance, and changes in the distance, of an object) decreases with age (Bell, Wolfe, and Bernholtz, 1972; Henderson and Burg, 1973, 1974; Shinar and Eberhard, 1976). One study found that the angle of stereopsis (seconds of visual arc) required for a group of drivers age 75 and older to discriminate depth using a commercial vision tester was roughly twice as large as that needed for a group of drivers ages 18 to 55 to achieve the same level of performance (Staplin, Lococo, and

Sim, 1993). McKnight and Stewart (1990) reported that the inability to judge gaps is not necessarily associated with a high crash rate to the extent that drivers can compensate for their deficiencies by accepting only inordinately large gaps. This tactic has a negative impact on operations as traffic volumes increase, however, and may not always be a feasible approach.

Judging in-depth motion is made difficult by the fact that when no lateral displacement occurs, the primary depth cue is the expansion or contraction of the image size of the oncoming vehicles (Hills, 1980). Studies of crossing-path crashes (where gap judgments of oncoming vehicle speed and distance are critical, as in passing situations) indicate an age-related difficulty in the ability to detect angular movement. In laboratory studies, older persons required significantly longer to perceive that a vehicle was moving closer (Hills, 1975). Staplin and Lyles (1991) reported research showing that, relative to younger drivers, older ones underestimate the speed of approaching vehicles. Similarly, Scialfa, Guzy, Leibowitz, Garvey, and Tyrrell (1991) showed that older adults tend to overestimate approaching vehicle velocities at lower speeds and underestimate at higher speeds, relative to younger adults. Older persons also apparently accept a gap to cross in front of an oncoming vehicle that is a more or less constant distance, regardless of the vehicle's speed. Analyses of judgments of the "last possible safe moment" to cross in front of an oncoming vehicle showed that older men accepted a gap to cross at an average constant distance, whereas younger men allowed a constant time gap and thus increased distance at higher speeds (Hills and Johnson, 1980). A controlled field study showed that older drivers waiting (stationary) to turn left at an intersection accepted the same size gap regardless of the speed of the oncoming vehicle (48 km/h and 96.5 km/h [30 mi/h and 60 mi/h]), while younger drivers accepted a gap that was 25-percent larger for a vehicle traveling at 96.5 km/h (60 mi/h) than their gap for a vehicle traveling at 48 km/h (30 mi/h) (Staplin et al., 1993).

The minimum passing sight distances listed in table 3B-1 of the *MUTCD* (FHWA, 2000) for marking passing zones are shorter than AASHTO's minimum passing sight distance values for the design of two-lane highways, as listed in table III-5 of the *Green Book* (AASHTO, 1994). Although the minimum passing sight distances specified by AASHTO are based on observations of successful car-passing-car observations, Hughes et al. (1992) commented that the model does not take into account the abortive passing maneuver, nor does it consider the length of the impeding vehicle. Saito (1984) determined that the values specified by the *MUTCD* for minimum passing distance are inadequate for the abortive maneuver, while Ohene and Ardekani (1988) asserted that the *MUTCD* sight-distance requirements are adequate for the driver to abort if the driver decelerates at a rate of 3.2 m/s^2 for a 64-km/h passing speed (10.5 ft/s^2 for a 40-mi/h passing speed) and at a rate of 3.9 m/s^2 for a passing speed of 80 km/h (12.8 ft/s^2 for a 50-mi/h passing speed). In any event, it cannot be assumed that drivers will always use the maximum acceleration and deceleration capabilities of their vehicles, particularly older drivers.

Consistent with the AASHTO operational model (AASHTO, 1994), passing sight distance is provided only at places where combinations of alignment and profile do not require the use of crest vertical curves. For horizontal curves, the minimum passing sight distance for a two-lane road is about four times as great as the minimum stopping sight distance at the same speed (AASHTO, 1994). By comparison, the *MUTCD* defines passing sight distance for vertical curves as the distance at which an object 1.07 m (3.50 ft) above the pavement surface can be seen from a point 1.07 m (3.50 ft) above the pavement. For horizontal curves, passing sight distance is defined by the *MUTCD* as the distance measured along the centerline between two points 1.07 m (3.50 ft) above the pavement on a line tangent to the embankment or other obstruction that cuts off the view of the inside curve (*MUTCD*, 2000). The length of the passing zones or the minimum distance between successive no-passing zones is specified as 120 m (400 ft) in the *MUTCD*. As Hughes, Joshua, and McGee (1992) pointed out, the *MUTCD* sight-distance requirements were based on a "compromise between a delayed and a flying passing maneuver, traceable back to the AASHTO 1940 policy that reflected a compromise distance based on a passing maneuver such that the frequency of maneuvers requiring shorter distances was not great enough to seriously impair the usefulness of the highway."

The basis for the minimum length of a passing zone (120 m [400 ft]) is unknown, however, because research has indicated that for design speeds above 48 km/h (30 mi/h), the distance required for one vehicle to pass another is much longer than 120 m (400 ft) (Hughes et al., 1992). Weaver and Glennon (1972) reported that, in limited studies of short passing sections on main rural highways, most drivers do not complete a pass even within a 244-m (800-ft) section; and use of passing zones remains very low when their length is shorter than 274.3 m (900 ft). Not surprisingly, it has been mentioned in the literature (Hughes et al., 1992) that the current AASHTO and *MUTCD* passing sight distance values are probably too low. Several studies have indicated that both the *MUTCD* and AASHTO passing sight distances are too short to allow passenger cars to pass trucks and for trucks to pass trucks (Donaldson, 1986; Fancher, 1986; Khasnabis, 1986).

Several research studies have been performed that have established and evaluated passing sight distance values for *tangent* sections of highways. As early as 1934, the National Bureau of Standards measured the time required for passing on level highways during light traffic and found that the time to complete the maneuver always ranged between 5 and 7 s, regardless of speed. Passing maneuvers were observed at speeds ranging from 16 to 80 km/h (10 to 50 mi/h). They concluded that 274.3 m (900 ft) of sight distance was required for passing at 64 km/h (40 mi/h). Harwood and Glennon (1976) reported that drivers are reluctant to use passing zones less than 268 m (880 ft). They recommended that design and marking standards should be identical and include both minimum passing sight distances and minimum length of passing zones, with minimum passing sight distance values falling between the AASHTO and *MUTCD* values. Kaub (1990) presented a substantial amount of data on passing maneuvers on a recreational two-lane, two-way highway in northern Wisconsin. Under low and high traffic volumes, he found that 24–35 percent and 24–50 percent of all passes, respectively, were attempted in the presence of an opposing vehicle; the average time in the opposing lane at (96 km/h [60 mi/h]) was 12.2 s under low traffic volumes and 11.3 s with high traffic volumes.

Passing lanes, also referred to as overtaking lanes, are auxiliary lanes provided on two-lane highways to enhance overtaking opportunities. Harwood, Hoban, and Warren (1988) reported that passing lanes provide an effective method for improving traffic operational problems resulting from the lack of passing opportunities, due to limited sight distance and heavy oncoming traffic volumes. In addition, passing lanes can be provided at a lower cost than that required to construct a four-lane highway. Based on Morrall and Hoban (1985), the design of overtaking lanes should include advance notification of the overtaking lane; a KEEP RIGHT UNLESS OVERTAKING sign at the diverge point; advance notification of the merge and signs at the merge; and some identification for traffic in the opposing lane that they are facing an overtaking lane. They reported that there is general agreement that providing short overtaking lanes at regular spacing is more cost-effective than providing a few long passing lanes. This feature becomes increasingly attractive as the diversity of driving styles and driver capability levels grows, with more aggressive motorists accepting greater risks to overtake slower moving vehicles.

Finally, the benefits of fluorescent retroreflective sheeting for increased daytime and nighttime conspicuity are reported by Jenssen, Moen, Brekke, Augdal, and Sjøhaug (1996). They conducted a controlled field study of daytime and nighttime visibility performance of fluorescent and non-fluorescent yellow traffic signs, both fabricated with retroreflective sheeting that provides for high brightness at wide observation angles (ASTM D4956-01,Type IX). The subjects included younger (ages 18-25) and older (ages 55-75) drivers. Under daytime conditions, the fluorescent yellow signs with Type IX sheeting provided a 90-m (295-ft) increase in sign shape recognition distance over the non-fluorescent yellow signs with Type IX sheeting for the older driver sample, and a 57-m (187-ft) increase for the younger driver sample. At a speed of 100 km/h (62 mi/h), this additional detection distance would translate to 3.2 s of extra reaction time for the older drivers and 2.1 s of extra reaction time for the younger drivers. At nighttime, the signs fabricated with Type IX sheeting provided an additional sign shape recognition distance of 288 m (945 ft) over signs fabricated with engineering grade sheeting (Type I), and an additional 149 m (489 ft) of shape recognition distance over signs made with high-intensity sheeting (Type III) for the older driver sample. The younger driver sample performed similarly, with increased sign shape recognition distances for the signs made with Type IX sheeting (308 m [1010 ft] over the signs made with Type I engineering grade sheeting, and 147 m [482 ft] over the signs fabricated with Type III high-intensity sheeting). These increased distances translate to an additional 5 to 10 s of reaction time at a speed of 100 km/h.

The age differences in driver capability and behavior noted earlier—i.e., age-related difficulties in judging gaps and in increased perception-reaction time, coupled with slower driving speeds—support a recommendation for use of the more conservative passing sight distance values specified by AASHTO (1994). In addition, a raised treatment to improve drivers' preview of the end of a passing zone—the widely recognized NO PASSING ZONE pennant, either oversized or fabricated with fluorescent yellow retroreflective sheeting that provides for high brightness at wide observation angles (e.g., Type IX) for added daytime and nighttime conspicuity—can reasonably be expected to facilitate older drivers' decisions and responses in situations where safe operations dictate that they should abort a passing maneuver. Finally, a recommendation to implement passing/overtaking lanes may be justified in terms of overall system safety and efficiency.

IV. CONSTRUCTION/WORK ZONES

The following discussion presents the rationale and supporting evidence for Handbook recommendations pertaining to these five design elements (A–E):

A. Lane Closure/Lane Transition Practices
B. Portable Changeable (Variable) Message Signing Practices
C. Channelization Practices (Path Guidance)
D. Delineation of Crossovers/Alternate Travel Paths
E. Temporary Pavement Markings

A. Design Element: Lane Closure/Lane Transition Practices

Table 42. Cross-references of related entries for lane closure/lane transition practices.

Applications in Standard Reference Manuals		
***MUTCD* (2000)**		***Traffic Engineering Handbook* (1999)**
Sect. 6C.05 Sect. 6C.06 Sects. 6F.15 & 6F.16 Sects. 6F.69 through 6F.72 Sect. 6F.53 Sect. 6F.55 Sect. 6G.04 Sect. 6G.09	Sects. 6G.11 through 6G.18 Tables 6C-1 & 6C-2 Figs. 6H-3, 6H-5, 6H-6, 6H-10 through 6H-12, 6H-15, 6H-16, 6H-18, 6H-19, & 6H-21 through 6H-46 plus associated notes with each figure.	Pp. 419-423, Sect. on *Retroreflectivity* P. 425, Sect. on *Changeable Message Signs* P. 439, Sect. on *Lane Reduction Transitions* P. 442, Sects. on *Vertical Panels & Drums*

Minimum requirements for safely negotiating a lane closure include an awareness of a decrease in pavement width ahead, and of the direction of the lateral shift in the travel path; a detection of traffic control devices marking the location of the lane drop (beginning of taper); a timely decision about the most appropriate maneuver, taking other nearby traffic into account; and smooth vehicle control through maneuver execution. In the vicinity of a lane closure, the longer the information needs supporting these requirements remain unmet for the *least capable* drivers within the traffic stream, the less likely is a smooth transition through the work area for *all* drivers (Goodwin, 1975). The more time that is required for older drivers to prepare and initiate a merging maneuver, the more dense following traffic (including the adjacent lane) is likely to become; this, in turn, will make gap judgments and maneuver decisions at the point of a lane closure more difficult, and will increase the likelihood of erratic vehicle movements resulting in conflicts between motorists.

Relevant alterations in older adults' cognitive-motor processes include: failure to use advance preparatory information (Botwinick, 1965); difficulty in processing stimuli that are spatially incompatible (Rabbitt, 1968); initiation deficit in dealing with increased task complexity (Jordan and Rabbitt, 1977); and inability to regulate performance speed (Rabbitt, 1979; Salthouse, 1979; Salthouse and Somberg, 1982). Stelmach, Goggin, and Garcia-Colera (1987) found that older adults showed disproportionate response slowing when compared with younger subjects when there was low expectancy for a required movement. When subjects obtained full information about an upcoming response, reaction time (RT) was faster in all age groups. Stelmach et al. (1987) concluded that older drivers may be particularly disadvantaged when they are required to initiate a movement in which there is no opportunity to prepare a response. Preparatory intervals and length of precue viewing times are determining factors in age-related differences in movement preparation and planning (Goggin, Stelmach, and Amrhein, 1989). When preparatory intervals are manipulated such that older adults have longer stimulus exposure and longer intervals between stimuli, they profit from the longer inspection times by performing better and exhibiting less slowness of movement (Eisdorfer, 1975; Goggin et al., 1989). Since older drivers benefit from longer exposure to stimuli, Winter (1985) proposed that signs should be spaced farther apart to allow drivers enough time to view information and decide which actions to take. Increased viewing time will reduce response uncertainty and decrease older drivers' RT.

In focus group discussions consisting of 81 drivers ages 65 to 86, pavement width transitions were identified as sources of difficulty by 50 percent of the participants (Staplin, Harkey, Lococo, and Tarawneh, 1997). The drivers participating in these discussions suggested longer merging areas to give them more opportunity to find a safe gap and the use of multiple warning signs to allow them to plan their maneuver at an earlier point upstream. Use of multiple signs to give advance notice of downstream work zones and of required maneuvers was also offered as a desired change by older drivers participating in an earlier focus group (Staplin, Lococo, and Sim, 1990).

Lyles (1981) conducted studies on two-lane rural roads to evaluate the effectiveness of alternate signing sequences for providing warning to motorists of construction and maintenance activities that required a lane closure. The signs tested included a standard *MUTCD* warning sequence, the same sequence augmented with continuously flashing warning lights on the signs, and a sequence of symbol signs (WORKER and RIGHT LANE CLOSED). The most effective sign sequence was one that was flasher-augmented; this treatment was twice as effective as similar signs with no warning lights in slowing vehicles in the vicinity of the lane closure.

The use of word messages on signs in highway work areas raises sign legibility issues for older drivers. In research conducted to improve the legibility of the RIGHT/LEFT LANE CLOSED and ROAD CONSTRUCTION series signs using test subjects in three age groups (18–44, 45–64, and 65 and older), Kuemmel (1992) concluded the following for black-on-orange (negative contrast) signs: (1) signs that increased both letter size and stroke width (SW) while maintaining or increasing the standard alphabet letter series resulted in the best improvement; (2) increasing letter size while decreasing the alphabet series (e.g., from C to B) reduces sign legibility, particularly at night; (3) the use of letter series E, with its 21-percent increase in SW-to-letter height over 200-mm (8-in) series C letters, appears to overcome the problems of irradiation (or the overglow phenomenon) with high-intensity retroreflective materials, thus increasing night legibility; (4) the legibility distance of the ROAD CONSTRUCTION signs can be increased by changing the word CONSTRUCTION to WORK, and increasing the letter size from 175-mm (7-in) series C to 200-mm (8-in) series C; and (5) for the RIGHT LANE CLOSED series, use of symbol signs will have to supplement word legend signs, and for the CENTER LANE CLOSED series, redundant signs will have to be employed if a 1200-mm (48-in) maximum sign size is to be maintained. The author pointed out that the minimum required visibility distance (MRVD) for both signs is 101 m at 88 km/h and 112 m at 104 km/h (331 ft at 55 mi/h and 369 ft at 65 mi/h). The legibility distances obtained in this study for the current standard construction work-zone signs ranged from 198 m (650 ft) for the best observers to 43 m (140 ft) for the worst observers. In addition, 85th percentile values were closer to the minimum legibility distances than they were to the maximum legibility distances. This finding reinforces the need for redundant signing during the approach to a work zone.

Recent studies have centered on the use of fluorescent orange signs for work-zone applications, particularly as their increased conspicuity may benefit older drivers with diminished visual capabilities by providing longer detection distances. Jenssen, Moen, Brekke, Augdal, and Sjøhaug (1996) state that flourescent materials have the potential to increase daytime conspicuity through increased contrast, while the nighttime brightness is sustained through a microprismatic retroreflective system. Burns and Pavelka (1995) explain that the high visibility of fluorescent materials is due to their ability to absorb energy in the near ultraviolet and visible region of the electromagnetic spectrum, and then to re-emit the energy as longer wavelength, visible light. Conventional colorants don't have this property. During the daylight hours from dawn to dusk, there is always sufficient solar energy to elicit light emission from fluorescent materials, irrespective of the cloud cover. Therefore, fluorescent colors maintain a significant daytime visibility advantage over ordinary colors in all types of weather.

Jenssen et al. (1996) conducted a controlled field study using 35 younger subjects (ages 18 to 25) and 44 older subjects (ages 55 to 75) to compare the detection distance, color recognition distance, and the legibility distance of fluorescent signs with traditional signs under daytime and nighttime conditions. In this study, subjects sat in an open-ended container on a railway car that moved at a speed of 15 km/h (9 mi/h) along a set of unused railroad tracks. Subjects used a response form and were trained to look for specific signs. They activated a response lever that sent a signal to a distance measuring computer and then recorded what they observed in the categories provided on their response forms for sign detection, shape, color, symbol, and text. The signs of interest for this discussion were those with an orange background and black text.

Signs with fluorescent orange Type VII retroreflective sheeting were compared to signs with standard orange Type VII retroreflective sheeting, signs with standard orange Type III high-intensity grade retroreflective sheeting, and signs with standard orange Type I engineering-grade retroreflective sheeting. During the day, only signs with Type VII optics were used. The Norwegian town names LENSVIK, LAKSVIK, or LEKSVIK, appeared in randomized order on the signs. The height, angle, and distance of the signs relative to the track were adjusted according to standards for Norwegian two-lane roadways to ensure realistic viewing positions. Signs were always placed on the right side of the track. For nighttime trials, original headlights for a Volkswagen Golf type 1 vehicle (placed on the railcar at the standard vehicle headlight orientation) were used.

Detection, shape recognition, color recognition, and content recognition were accomplished at significantly greater distances for fluorescent orange retroreflective signs than for the standard orange retroreflective signs for both younger and older subjects under daytime conditions. The mean detection distance for all subjects during daytime conditions for the fluorescent orange retroreflective signs was 822 m (2697 ft), compared to 783 m (2569 ft) for standard orange retroreflective signs. This 40-m difference in detection distance was statistically significant. The difference in mean detection distance was larger for the older subjects (50 m) than for the younger subjects (22 m); however, both age groups demonstrated significantly longer detection distances when viewing the fluorescent orange retroreflective signs. The mean shape recognition distance across all subjects during the day was 744 m (2441 ft) for the fluorescent orange retroreflective signs and 651 m (2136 ft) for the standard orange retroreflective signs. Younger subjects were able to correctly recognize the shape of fluorescent signs at an average distance that was 100 m longer than for the standard signs, and older subjects demonstrated an average shape recognition distance difference of 59 m. Fluorescent signs also showed significantly longer correct color recognition distances (584 m [1916 ft] across age groups) than standard signs (469 m [1539 ft] across age groups). Color recognition distances were 130 m longer for younger subjects and 106 m longer for older subjects when viewing the fluorescent signs during the day compared to the standard signs. In terms of legibility distances during the day, across all subjects, the fluorescent signs significantly outperformed the non-fluorescent signs, with a difference in mean legibility distance of 13 m (43 ft).

At night, there were no significant differences in detection, shape recognition, color recognition, or contents recognition distances between fluorescent orange retroreflective signs with Type VII sheeting and standard (non-fluorescent) orange retroreflective signs with Type VII sheeting for either age group. However, comparisons between the three types of retroreflective sheeting indicated that the signs with Type VII sheeting produced detection distances that were 42 m (138 ft) longer than the signs with high-intensity grade sheeting and 62 m (203 ft) longer than the signs with engineering-grade sheeting for the older drivers. For the younger drivers, detection distances for the signs with Type VII sheeting were 19 m (62 ft) longer than those produced by the signs with high-intensity grade sheeting and 36 m (118 ft) longer than those produced by the signs with engineering-grade sheeting.

The mean sign detection distance, shape recognition distance, color recognition distance, and contents recognition distance are presented in table 43 as a function of subject age group and lighting condition (day versus night) for the signs with standard orange Type VII sheeting and for the signs with fluorescent orange Type VII sheeting.

Burns and Pavelka (1995) conducted a field study using 14 drivers ages 19 to 57 (median age: 40 years) to compare the visibility and conspicuity of durable retroreflective fluorescent sheetings (orange, red, yellow, and yellow-green) with the same color standard highway sheeting (orange, red, yellow, yellow-green, and green) at midday and at dusk. Circular targets with an area measuring 9.3 cm^2 (0.01 ft^2) were viewed in pairs (one fluorescent and one standard highway color sign) against a 1.2-m by 1.2-m (4-ft by 4-ft) background. The background consisted of a complex camouflage pattern made up of 23-percent light green, 34-percent green, 25-percent brown, and 18-percent black. The targets were placed 0.3 m (1 ft) apart and were viewed at four distances during the day (120 m [394 ft], 90 m [295 ft], 60 m [197 ft], and 30 m [98 ft]). At dusk (15 min before sunset and 15 min after sunset), signs were viewed only at 30 m. Subjects viewed the target pairs while seated in a vehicle that had the headlights turned off. An electronic shutter system provided a viewing duration of 2 s. After each target pair was viewed, subjects provided responses to indicate: (1) the number of targets detected (0, 1, or 2); (2) target location (left or right); (3) target color (left color and right color); and (4) attention-getting value (was one target perceived more easily, or did one target attract your attention more than the other?).

Table 43. Subject performance as a function of sheeting type (fluorescent orange Type VII versus standard orange Type VII) and as a function of subject age group and lighting condition. Source: Jenssen, Moen, Brekke, Augdal, and Sjøhaug (1996).

Lighting Condition	Age Group	Sign Color (Type VII Sheeting)	Mean Distance (m)			
			Sign Detection	Shape Recognition	Color Recognition	Contents Recognition
Daytime	Young	Standard Orange	828	707	518	123
		Fluorescent Orange	851	808	648	131
	Old	Standard Orange	726	591	410	92
		Fluorescent Orange	776	650	516	108
Nighttime	Young	Standard Orange	838	707	592	88
		Fluorescent Orange	815	623	548	93
	Old	Standard Orange	771	589	471	92
		Fluorescent Orange	758	555	430	66

During the day, the durable fluorescent targets evaluated in the Burns and Pavelka (1995) study were detected with a higher frequency (close to 100 percent) and at greater distances than the standard highway colors. At midday (facing north on an overcast day), from a distance of 120 m, 93 percent of the subjects were able to detect the fluorescent orange targets; however, only 43 percent of the subjects could detect the standard orange targets at this distance under the same lighting conditions. At 90 m, 100 percent of the subjects detected the fluorescent signs, compared to 92 percent who detected the standard orange signs. At dusk (15 min after sunset), at a distance of 30 m, the fluorescent orange signs were detected by 96 percent of the subjects and the standard orange signs were detected by 85 percent of the subjects.

The fluorescent signs also showed greater color recognition than the standard highway signs at all distances. During midday (overcast facing north), the fluorescent orange signs were correctly identified by 61 percent of the subjects at 120 m, 58 percent of the subjects at 90 m, 86 percent of the subjects at 60 m, and 82 percent of the subjects at 30 m. By comparison, the standard orange signs were identified correctly by 7 percent, 23 percent, 64 percent, and 93 percent of the subjects at the same distances, respectively. At dusk (15 min before sunset), at 30 m, the fluorescent orange signs were correctly identified as orange by 74 percent of the subjects, compared to 62 percent of the subjects for the standard orange signs.

The fluorescent orange sign in each pair of viewings was subjectively determined to be more conspicuous (more attention-getting) than the standard orange highway sign at 30 m, under all lighting conditions (midday, 15 min before sunset, and 15 min after sunset). Luminance measurements were taken of the targets and their backgrounds, so that a contrast ratio could be calculated. The fluorescent signs always produced a higher contrast ratio than the standard signs. The contrast ratios for the fluorescent orange and standard orange signs are shown in table 44 below, under the various natural lighting conditions utilized in the study.

Table 44. Luminance contrast ratio ($[L_t-L_b]/L_b$) under different lighting conditions for fluorescent orange and standard highway orange signs. Source: Burns and Pavelka (1995).

Color	**Sign Direction and Lighting Condition**		
	Facing South Midday-Clear	**Facing North Midday-Overcast**	**Facing North Dusk-Overcast**
Fluorescent Orange	5.4	4.4	1.0
Standard Orange	1.8	2.0	0.5

The authors conclude that fluorescent orange signs are more conspicuous than standard highway orange sign colors during the day (as were the other fluorescent colors); are detected with higher frequency; and are recognized with greater accuracy at farther distances. Fluorescent signs provide a greater contrast with the background scene and, therefore, should be considered as a countermeasure to address problems that older drivers have in the detection and recognition of traffic signs when viewed against a cluttered background.

Finally, Hummer and Scheffler (1999) conducted a field study at seven long-term work zones in North Carolina with left lane drops on multi-lane highways to determine whether the increase in the conspicuity of fluorescent orange signs leads to positive operational changes in driver behavior. All seven sites were left lane drops on four-lane highways (with standard lane and shoulder widths), with the following sequence of orange signs (in pairs, with one sign on each side of the highway):

- Two fluorescent BEGIN WORK ZONE text message signs located 0.6 to 1.9 km (0.4 to 1.2 mi) from the taper.
- Two text message LEFT LANE CLOSED AHEAD signs located 0.4 to 1.1 km (0.25 to 0.68 mi) from the taper.

- Two symbol message LEFT LANE CLOSED AHEAD signs located 0.16 to 0.5 km (0.1 to 0.31 mi) from the taper.
- Two text message LEFT LANE CLOSED signs at the beginning of the taper.

Six sites had 90-km/h (55-mi/h) speed limits and wide grassy medians, and one site had a 105-km/h (65-mi/h) speed limit. Before the study was conducted, both treatment and control sites existed as work zones with standard orange signs, except for the first sign in the pair, which was fluorescent. In the "before" period, five operational measures were collected on this standard set of signs. In the "after" period, the standard orange signs were replaced with fluorescent signs (same size and message) at the treatment sites, and the same measures were recorded at these treatment sites, as well as at the control sites where the standard signs were left in place. Two weeks elapsed before data were collected in the "after" period, to eliminate novelty effects. The operational measures included: (1) traffic conflicts (one vehicle brakes or swerves to avoid hitting another); (2) percentage of all vehicles in the left lane; (3) percentage of trucks in the left lane; (4) mean speed; and (5) speed variance. These data were collected at the beginning of the taper, at the BEGIN WORK ZONE sign, and at the midpoint of the approach.

With regard to traffic conflicts, a reduction from 153 to 136 at the treatment sites (with fluorescent signs) was observed from the before period to the after period, while an increase from 160 to 187 was observed at the control sites (with standard signs) from the before period to the after period. This reduction in conflict frequency was statistically significant when sites without potential confounding factors were removed from the analysis. Regarding the number of vehicles in the left lane, there was a significant reduction in the percentage of vehicles at the midpoint of the work-zone approach at the sites with fluorescent signs (more than a 5-percent reduction, or 100+ fewer vehicles); similarly, an increase in the percentage of trucks that moved out of the left lane before the midpoint (16 more trucks, or 30 percent more than expected) and at the taper (12 more trucks than expected) was observed at the sites with fluorescent signs, compared to sites with the standard orange signs.

Differences in mean speeds were not statistically significant; speeds increased by approximately 1.6 km/h (1 mi/h) at the midpoint and taper of treatment sites, and decreased by the same amounts at the control sites. However, speed variance decreased at the midpoint and at the taper of the treatment sites (with fluorescent signs), relative to speeds monitored at the control sites (with standard orange signs).

Hummer and Scheffler (1999) state that the operational changes documented during their study would translate to fewer collisions in work zones that display fluorescent orange signs compared to those that display the standard orange signs. Although the overall reduction in traffic conflicts in this study was small (approximately 7 percent), they recommend that agencies use fluorescent orange sheeting on warning signs in work zones similar to those studied, as well as for work zones where warning drivers is as critical or more critical than it was in the current study. These would include long-term work zones where there is flagging, temporary traffic signals, sharp lane shifts, and changed merging patterns, as well as in many temporary and moving work zones. Hummer and Scheffler (1999) state that fluorescent orange sheeting costs only a few dollars more per sign installation than standard orange sheeting, and even if work-zone collision frequencies declined by only 1 or 2 percent, the benefit-to-cost relationship would be substantial.

Next, a number of studies have been performed to determine the effectiveness and motorist comprehension of a static sign and a changeable message sign (CMS)—also referred to as a variable message sign (VMS)—for lane closures. A general indication of the importance of the CMS to accomplishing lane control in advance of work zones is provided by a field study on a four-lane section of I-35 in San Antonio, TX conducted by Dudek, Richards, and Faulkner (1981) to evaluate the effects of CMS messages on lane changes at a work-zone lane closure. The measure of effectiveness used to evaluate the CMS was the percentage of vehicles that remained in the closed (median) lane as traffic progressed toward the cone taper. The results indicated that the CMS did encourage drivers to vacate or avoid the closed lane, compared with driver response at the same site without use of the CMS. The percent volumes in the closed lane were significantly lower when a lane-closure message was displayed than during periods when the sign was blank. Specifically, there was a 46-percent greater reduction in the lane volume attributable to the CMS.

During the conduct of field studies for NCHRP project 3-21(2), the relative proportions of traffic in the through and closed lanes approaching construction lane closures were observed for a sample of more than 196,500 vehicles (Transportation Research Board, 1981). Data gathered in California, Colorado, and Georgia were used to compare these lane distributions between baseline (no CMS) conditions and various CMS applications. A fourth data set, gathered in South Carolina, was used to determine the relative effects between certain CMS message alternatives (i.e., speed and closure, speed and merge, closure and merge advisories) and various placement configurations (i.e., one CMS at 610 m [2000 ft] in advance; one CMS at 1207-m [3960-ft] advance placement; two CMS devices, one at each advance location; or one CMS placed at 1207 m [3960 ft] in advance of the taper and an additional arrow panel at the 610-m [2000-ft] location). Findings indicated increased preparatory lane-change activity, smoother lane-change profiles, and significantly fewer "late exits" (exit from a closed lane within 30.5 m [100 ft] of closure) in locations where a CMS was applied at the 1207-m (3960-ft) advance location and an arrow panel at the 610-m (2000-ft) location.

Additional studies of flashing arrow panels at construction sites have shown that they are effective in shifting approaching traffic out of a closed lane (Bates, 1974; Shah and Ray, 1976; Graham, Migletz, and Glennon, 1978; Bryden, 1979; Faulkner and Dudek, 1981). These studies found that arrow panels were effective because they promote early merging into the open lane and fewer vehicles remained in the closed lane at the start of the lane-closure taper. A basis thus exists to assert that a CMS used to give advance notice of the need to exit a lane, followed by the application of an arrow panel, would be of clear benefit to drivers with diminished capabilities resulting from aging, inattentiveness, or transient impairment (e.g., due to fatigue, alcohol, or drugs). While the specific location of the arrow panel in this approach should be consistent with the signing sequence indicated in *MUTCD* Part 6H (figure 6H-33 for divided highways), placement at the beginning of the taper is suggested by the findings reported above.

Mace, Finkle, and Pennak (1996) conducted a static and a dynamic field study to determine the minimum and optimum lamp intensities needed for arrow panel legibility (left arrow or chevron vs. right arrow or chevron presentation) during the day, and minimum and maximum intensity for nighttime operations to minimize glare effects. The authors cite the work of Faulkner and Dudek (1982), who found that sight distances to arrow panels (APs) influence driver behavior, such that when APs are used too far in advance of a lane closure (e.g., 1219 m [4000 ft], drivers tend to return to a vacated lane. Also, if sight distance is less than 457 m (1500 ft), an advance supplemental AP is desirable. While no data exist to document problems in the safe use of APs by seniors, the following recommendations (see table 45) suggested by Mace et al. (1996) for arrow panel lamp intensity provide a useful reference for practitioners. These values ensure visibility for decision sight distances (DSDs) of 457 m and 283 m (1500 ft and 930 ft), for high-speed and low-speed roadways, respectively.

Table 45. Recommended minimum on- and off-axis lamp intensities of arrow panels to ensure daytime visibility for older drivers and the maximum intensity recommended for nighttime operations to ensure safe levels of discomfort and disability glare for older drivers for high-speed (≥73 km/h [45 mi/h]) and low-speed (<73 km/h [45 mi/h]) roadways.
Source: Mace, Finkle, and Pennak (1996).

Situation	Luminous Intensity Requirements (cd per lamp)		
	Minimum On-Axis	**Minimum Off-Axis**	**Maximum Hot Spot**
Low-Speed Day	300	60	NA*
High-Speed Day	500	100	NA
Low-Speed Night	90	18	370
High-Speed Night	150	30	370

A questionnaire was also completed during the conduct of NCHRP project 3-21(2) by 489 subjects ranging in age from under 20 to 80 to gather measures of driver detection, recognition, and comprehension of the CMS devices. Twenty percent of the drivers were age 60 and older. The five tested message conditions were: (1) speed and closure advisory (MAX SPEED 45 MPH/RIGHT LANE CLOSED); (2) speed and merge advisory (MAX SPEED 45 MPH/MERGE LEFT); (3) merge and closure advisory (RIGHT LANE CLOSED/MERGE LEFT); (4) speed advisory only (SLOW TO 45 MPH); and (5) closure advisory only (RIGHT LANE CLOSED AHEAD). Drivers consistently reported that the speed advisory and lane closure message combination was most helpful, was the easiest to read, best met their information needs, and would be most likely to cause them to change lanes early and reduce speed.

A recent human factors laboratory study was conducted to determine which CMS message alternatives would be most likely to enhance motorists' compliance with lane-control messages in work zones (Gish, 1995). The subjects were divided into two age groups consisting of 24 subjects each, the youngest drivers had a mean age of 23.1 years (range: 16–33), and the oldest drivers had a mean age of 70.2 years (range: 65–84). The results of this study indicated that older drivers were more likely to reduce their speed and change lanes than the younger drivers, and that both older and younger drivers' compliance with lane-change messages was strongly influenced by surrounding vehicles and by the visibility of the lane closures themselves, which exerts a strong influence on message credibility. Other factors, such as traffic density, static displays, and merge arrows (arrow panels), influence driver compliance with CMS messages. To optimize lane-change compliance, Gish (1995) recommended that static displays, merge arrows, and other devices be used in addition to CMS messages. A need to study the long-term effectiveness on non-standard messages was also indicated, and potential improvements in work-zone safety and operations through the use of condition-responsive (real-time) traffic control systems that provide continuously updated information to motorists (for enhanced credibility) were identified.

B. Design Element: Portable Changeable (Variable) Message Signing Practices

Table 46. Cross-references of related entries for portable changeable (variable) message signing practices.

Applications in Standard Reference Manuals	
MUTCD (2000)	*Traffic Engineering Handbook* (1999)
Sects. 2E.21, 6F.52, & 6F.53 Figs. 6H-4, 6H-13, 6H-17, 6H-22, 6H-24, 6H-30, 6H-32 through 6H-35, 6H-37 through 6H-39, 6H-42, & 6H-44 plus associated notes with each figure.	P. 424, Para. 5 P. 425, Sect. on *Changeable Message Signs* P. 638, *Appendix H: Dynamic Message Sign Use Guidelines*

The effectiveness of a changeable message sign (CMS), gauged in terms of observable driver behaviors that traffic management procedures are designed to elicit, rests upon a set of reasonably well-understood human factors. A motorist information system must be rational, relevant, and reliable. Driver sensory/perceptual and cognitive capabilities must be thoughtfully considered to ensure that a message will be acquired and then understood, recalled, and applied by the driver within a desired time frame; the message must seem to clearly apply to the driver *and* to reflect current conditions to be credible; and it must be accurate in describing what the driver experiences downstream. The credibility of a highway advisory message certainly depends in part upon a presentation strategy that is rational, but it also must be perceived to be relevant to the individual motorist, and reliable to the point of being virtually error-free. Reliability requirements—being dependent on real-time data on operations as input to the traffic control system—are most difficult to meet, but probably the most important if high rates of compliance in drivers' vehicle control decisions are ever to be realized.

A motorist's ability to use highway information is governed by: (1) *information acquisition*, or how well the source can be seen or heard; and (2) *information processing*, or the speed and accuracy with which the message content can be understood, and its ease of recall by the motorist after message presentation is completed.

In the acquisition of CMS information a visual task the key factors are: (1) its *conspicuity*, or "attention-getting value" to the motorist; (2) the size, brightness (contrast), stroke width-to-height ratio, and spacing of individual characters of text, which together determine the *legibility* of the message; (3) the *placement* of the CMS device—overhead versus one side versus both sides of the highway—which affects its likelihood of being blocked from a motorist's view by other vehicles, as well as the "eyes away from the road" time required to fixate upon the message; and (4) the *exposure time*, or available viewing time, of each message phase presented on a CMS.

Conspicuity is generally not a problem for any type of CMS under low traffic volumes, although under high volumes with a significant mix of heavy vehicles, a motorist may fail to notice a roadside device because of obscuration. Good conspicuity is achieved by overhead devices under all conditions. The attention-getting value of a CMS display can be maximized by flashing operations, but this also works against information acquisition by reducing exposure time and legibility; this strategy is thus uniformly discouraged for an entire message. In rare circumstances, for a unit of information deemed particularly critical by the highway authority, the flashing of a single text element *within* a message at a slow rate may be justified. The use of flashing text may help bring the sign to the attention of an older driver who has a reduction in his/her useful field of view and may otherwise fail to notice the sign. If it is standard policy to leave the signs blank, then the mere display of a message will capture the driver's attention. However, if the CMS in question always has some type of message displayed, then *slowly* flashing (e.g., two cycles per phase) the problem statement line only may be warranted in order to attract attention. A preferred strategy under such circumstances would be to activate a flashing warning light separate from, though clearly attached to, the CMS.

The legibility of a CMS is influenced by the same factors influencing character and message legibility of static signs, including the key factor of driver visual performance capability. Letter acuity declines during adulthood (Pitts, 1982) and an older adult's loss of acuity is accentuated under conditions of low contrast, low luminance, and where there is crowding of visual contours (Sloane, Owsley, Nash, and Helms, 1987; Adams, Wong, Wong, and Gould, 1988). In any event, the legibility for a current CMS is determined primarily by the technology and the device configuration (number of rows; characters per row; and number, size, and spacing of pixels per character) as fabricated by a given manufacturer, and for all practical purposes can be treated as a fixed factor–modified by environmental considerations–in considering whether a particular system as implemented in the field will meet motorists' needs.

For any given speed, older drivers' needs dictate a legibility distance that permits the entire CMS message to be read *twice in its entirety*. As a general rule, at least 305 m (1000 ft) of legibility distance for a motorist with 20/40 visual acuity should be provided on an 88-km/h (55-mi/h) facility. Of the studies that assessed various character matrix forms (number of elements per character cell), most found a 7 x 9 element matrix to be necessary when using lowercase letters because of the descenders and ascenders; however, a 5 x 7 font was generally deemed to be acceptable with uppercase lettering only. The *MUTCD* specifies a minimum legibility requirement of 200 m (650 ft) under both daytime and nighttime conditions for portable changeable message signs. Given that the most common format for a portable sign is 450-mm- (18-in-) tall characters arranged in three lines of eight characters each, this provides for a legibility distance of 0.44 m/mm (36 ft/in) of letter height. Thus, letter sizes of at least 450 mm (18 in) should be used to accommodate older drivers' diminished visual acuity. Other variables found to significantly effect CMS legibility for older observers are font, letter width-to-height ratio, contrast orientation, letter height, case, and stroke width (Jenkins, 1991; Mace, Garvey, and Heckard, 1994). The most consistent finding across studies evaluating CMS design elements was that the results found for older drivers were quantitatively, but not qualitatively, different from those of their younger counterparts. In other words, if manipulation of a variable resulted in an improved score for younger observers, it almost invariably improved older observer performance as well.

Garvey and Mace (1996) conducted several laboratory and controlled field studies to determine the optimum legibility requirements of a CMS, particularly for older drivers. The laboratory studies included 24 "young" subjects ages 16 to 40 (mean age: 26.6); 25 "old" subjects ages 62 to 73 (mean age: 67.9); and 21 "old-old" subjects age 74 and older (mean age: 77.2).

The first laboratory study used a CMS simulator that was programmed on a PC-compatible computer, simulating nighttime viewing conditions. Only positive contrast signs (light letters against a dark background) were used. The objectives were: (1) to determine the optimum width-to-height ratio (W:H) and stroke width-to-height ratio (SW:H); (2) to identify the CMS font that produced the smallest size legibility thresholds; and (3) to determine the effect of color on legibility. Six different sizes of each sign were evaluated. The dependent variable in the study was the threshold size at which a character became legible, which was converted into a legibility index (LI) reported in m/cm (ft/in). Seven combinations of CMS matrix size, W:H, and SW:H combinations were evaluated to determine the optimum character legibility, as shown in table 47.

Table 47. Variables evaluated by Garvey and Mace (1996) using a CMS simulator to determine the optimum character legibility.

Matrix Size	Width-to-Height Ratio (W:H)	Stroke Width-to-Height Ratio (SW:H)
5 x 7	1.0	0.13
5 x 7	0.8	0.13
5 x 7	0.7	0.13
15 x 15	1.0	0.13
15 x 15	1.0	0.2
12 x 15	0.8	0.13
12 x 15	0.8	0.2

Results indicated that for all conditions, the younger group performed significantly better (smaller letter size required for legibility) than both older groups, and the "old" group performed better than the "old-old" group. The authors indicated that, generally, what worked well for one age group worked well for all ages. Across all age groups, increasing the width-to-height ratio (W:H) of a character from 0.7 to 1.0 increased the legibility index (LI) by 0.84 m/cm (7 ft/in). This provides an advantage of 38 m of legibility for the wider letter when using a 46-cm (18-in) letter height, or 1.5 s at 89 km/h (55 mi/h). A significant stroke width-to-height (SW:H) effect was also found. For the narrow letters (W:H = 0.8), a thinner stroke performed better than a wider stroke by 0.48 m/cm (5 ft/in). This effect was not significant with wider letters. There were no significant differences in legibility index as a function of matrix density. Therefore, for uppercase letters, increasing the number of elements beyond the standard 5 x 7 format did not improve legibility. The authors state that a typical CMS font with a W:H of 1.0 and a SW:H of 0.13 is optimal for the three age groups studied, from the median to the 85th percentile observer. Their data indicate that the 85th percentile "old-old" observer was capable of reading such a letter at the LI typically expected of a CMS (4.2 m/cm or 35 ft/in).

In another laboratory study using the same subjects and test apparatus, Garvey and Mace (1996) found that the fonts typically used by CMS manufacturers performed well, with the exception of double-stroke characters within a 5 x 7 character matrix. A double-stroke font provided an LI of 5.2 m/cm (43 ft/in) for young observers compared to 6.6 m/cm (57 ft/in) for the typical CMS font. For "old-old" observers, the double-stroke font provided an LI of 3.8 m/cm (32 ft/in) compared to the typical CMS font that provided 4.6 m/cm (38 ft/in). Across all age groups, the double-stroke font resulted in a decrement in LI of 1.2 m/cm (10 ft/in).

In the final laboratory study of CMS character legibility, Garvey and Mace (1996) found significant effects of contrast orientation on letter legibility. Positive-contrast stimuli (lighter colored letters on a dark background) produced an LI that was 1.4 m/cm (12 ft/in) higher than negative-contrast stimuli (dark letters on a lighter background). This improvement is equal to an additional 67 m (220 ft) of legibility distance for a 450-mm (18-in) letter height, or 2.75 s at 89 km/h (55 mi/h). White-on-black signs performed similarly to yellow-on-black signs, except for the highest percentile "old-old" group, where yellow-on-black signs were significantly better than white-on-black. Red-on-black signs performed as well as the other two colors for the young observers, but were found to be significantly less legible than yellow-or white-on-black signs for both groups of older observers. The authors point out that the reduced performance of the color red for older subjects is probably due to its lower luminance, and as people age, they become more sensitive to changes in target luminance. LI by sign color and observer age and percentile is shown in table 48.

In a dynamic field study, Garvey and Mace (1996) employed older and younger drivers to evaluate legibility distance and detection distance of six portable CMS's. Participants included 33 "young" subjects ages 19 to 40; 25 "old" subjects ages 59 to 72; and 26 "old-old" subjects ages 73 to 82. Other independent variables included contrast orientation (positive or negative); character height (450 mm [18 in] or 1050 mm [42 in]); lighting condition (backlit, frontlit, overcast, or rain); character luminance—day (350, 570, 850, or 1200 cd/m^2); character luminance—night (30, 80, 130, 200, 570, or 1200 cd/m^2); inter letter spacing—night (single or double); and sign lighting—night (internal versus external or backlight versus light-emitting diodes [LEDs]).

Significant findings in the field study included the following:

- At night, positive contrast messages (yellow on black) produced significantly longer legibility distances, representing a 29-percent improvement over negative contrast messages (black on yellow). The mean legibility distance for positive contrast messages was 152 m (497 ft), and the mean legibility distance for negative contrast messages was 118 m (386 ft). The "old-old" group showed significantly shorter legibility distances compared to the "young" and "old" groups, which were not significantly different from one another.

- Increasing luminance during the day up to 850 cd/m^2 produced significantly longer legibility distances; however, increasing the luminance from 850 to 1200 cd/m^2 did not significantly increase legibility distance. At night, the effects of increasing luminance were random, with the lowest and highest luminances both producing legibility distances of approximately 245 m (800 ft). Also, there was no significant interaction between character luminance and age group. [Note: Important guidance on procedures for valid measurement of CMS character luminance is provided by Garvey and Mace (1996)].

Table 48. Legibility index obtained by Garvey and Mace (1996) in a laboratory study of CMS character legibility as a function of driver age, sign color, and percentage of drivers accommodated.

Driver Age	Percentage Accommodated	Legibility Index (m/cm)		
		Yellow on Black	White on Black	Red on Black
16-40	50	7.4	7.4	7.6
	75	7.0	6.6	6.8
	85	5.8	6.2	6.4
	90	5.4	5.4	5.6
	95	4.8	5.2	4.2
62-73	50	6.4	6.6	5.8
	75	5.8	6.0	4.8
	85	5.6	5.2	4.6
	90	5.2	5.2	4.6
	95	4.2	5.0	3.8
74+	50	5.4	5.4	4.8
	75	5.0	4.6	4.0
	85	4.8	4.0	3.6
	90	4.2	3.8	3.4
	95	3.6	3.6	3.0

1 m/cm = 8.33 ft/in

Next, the target value, legibility, and viewing comfort of LEDs and fiber-optic CMS technologies were compared with flip-disk and conventional overhead guide signs in a field study conducted by Upchurch, Baaj, Armstrong, and Thomas (1991). Younger (ages 18 to 31) and older (ages 60 to 79) subjects in this study demonstrated mean daytime target values for fiber-optic, LED, and flip-disk technologies that were all significantly better (longer) than the values for conventional overhead signs. Under nighttime conditions, however, the poorest performance (shortest distances) were demonstrated by both age groups for the flip-disk technology, falling below the conventional sign values as well. The fiber-optic and LED signs again exceeded the conventional signs, based on nighttime mean target value, with the fiber-optic technology showing a slight superiority for older drivers. Under backlight (sun behind sign) and washout (sun behind driver) conditions, target values for all sign types decreased substantially and the differences among sign types diminished, but the fiber-optic technology still resulted in the best overall performance across age groups.

Legibility distance results tended to favor the conventional signs, followed by the fiber-optic signs, LED signs, and flip-disk technology. Mean daytime legibility distances for each sign type in this study were as follows: fiber-optic—0.74 m/mm (61 ft/in); LED—0.51 m/mm (42 ft/in); flip-disk—0.47 m/mm (39 ft/in); and conventional—1.07 m/mm (88 ft/in). Under nighttime conditions, the conventional signs again could be read at the longest mean distances, followed closely by the fiber-optic and LED signs, with the flip-disk technology showing the poorest performance. Backlit conditions favored the fiber-optic technology, and washout conditions favored the conventional signs; in both cases, however, the flip-disk technology resulted in the shortest legibility distances. Using a threshold for minimal acceptable legibility distance of 191 m (628 ft), the study concluded that flip-disk signs are deficient under all conditions *except* midday daytime viewing, LED signs are deficient under both backlit and washout sun conditions, and fiber-optic signs are deficient only with the sun glare present under backlit conditions.

Mean discomfort ratings were consistent with these patterns of results. Fiber-optic and conventional signs were assigned the best (lowest discomfort) ratings under daytime conditions by younger and older drivers alike. LED signs caused slightly more discomfort for older subjects, and flip-disk signs resulted in the highest discomfort ratings, especially for older drivers. Under nighttime conditions, only the flip-disk technology resulted in high discomfort ratings. Discomfort ratings were more even, and much higher, across sign types under backlit conditions where the sun was behind the sign, though flip-disk signs were still rated as being the worst by both age groups. Under washout conditions, subjects reported little discomfort with either the fiber-optic or conventional signs, but much greater and roughly equivalent levels of discomfort with the LED and flip-disk technologies.

Table 49 contains legibility distances from the Upchurch et al. (1991) study. For older drivers, the legibility distances are lower due to the well-documented degradation of visual performance with age. Unfortunately, this is the only study that has assessed legibility distances for older observers. The legibility distances for conventional bulb-matrix and LED/flip-disk hybrid CMS's were estimated from the results of the Upchurch data and data cited in Dudek (1991).

The older driver legibility distances in table 49 should be assumed to represent the legibility distances for the various types of technology represented. This ensures that the needs of older drivers have been met. The results suggest that a flip-disk CMS should not be used at night along roadways where average speeds reach or exceed about 88 km/h (55 mi/h).

Table 49. Daytime and nighttime predicted legibility distances for various sign technologies. Source: Upchurch et al. (1991).

Sign Technology (Character Height)	Daytime Legibility Distances		Nighttime Legibility Distances	
	Younger Observers	Older Observers	Younger Observers	Older Observers
Fiber-optic 400 mm (16 in)	307 m (1006 ft)	292 m (959 ft)	209 m (687 ft)	203 m (667 ft)
Light-emitting diodes 445 mm (17.8 in)*	247 m (812 ft)	208 m (681 ft)	242 m (794 ft)	183 m (602 ft)
Flip-disk 450 mm (18 in)	229 m (731 ft)	203 m (667 ft)	111 m (363 ft)	106 m (348 ft)
Bulb matrix 450 mm (18 in)	244 m (800 ft)	205 m (671 ft)	229 m (750 ft)	173 m (569 ft)
Hybrid LED/flip-disk 450 mm (18 in)	229 m (731 ft)	203 m (667 ft)	242 m (794 ft)	183 m (602 ft)

* Legibility distance of this technology decreases over time, because as LEDs age, they become less bright.

Although the bulb-matrix CMS was assessed by Upchurch et al. (1991), no legibility distances for that sign were reported. Legibility distances for this type of CMS have been obtained; however, it is unknown whether any older observers have been used in assessing legibility distances. Dudek (1991) cited a study in which bulb-matrix CMS's provided legibility distances of 244 m (800 ft) during the day and 229 m (750 ft) at night. These distances are similar to the legibility distances obtained by Upchurch et al. (1991) for an LED-type CMS using younger observers. Until psychophysical data can be obtained for older observers viewing bulb-matrix signs, the legibility distances for older observers are assumed to be roughly 204 m (671 ft) during the day and 173 m (569 ft) at night. These estimates are based on applying the ratio of older-to-younger legibility distances for the LED-type display.

There are also a number of hybrid CMS's that were not included in the Upchurch et al. study. A hybrid CMS applies the various combinations of sign technologies listed in table 49 within a single sign. Product literature for one manufacturer's hybrid LED/flip-disk sign states that the sign provides 274 m (900 ft) of legibility distance during the day and greater than 274 m (900 ft) at night, using character heights of 450 mm (18 in). Unfortunately, the methods used to obtain these legibility distances are unknown. Since the sign uses the reflective flip-disk technology during the day and LEDs at night, the legibility distances for older observers for the daytime flip-disk in table 49 (203 m [667 ft]) should be used as a more realistic estimate of legibility distance with LED/flip-disk hybrids. For nighttime viewing, use the nighttime LED legibility distance (183 m [602 ft]) in table 49.

CMS placement affects information acquisition under heavy traffic conditions where a center-lane driver's view of a roadside device may be obscured for lengthy intervals. If a facility has more than two lanes, consideration may be given to placement of a portable CMS in the median—space permitting and where glare from opposing vehicles is absent or minimal due to a large glare angle—rather than on the right shoulder, since lane control practices for heavy trucks are common throughout many corridors.

A motorist's reading time for a CMS message dictates the required exposure time at a given speed. Exposure time is the length of time a driver is within the legibility distance of the message. The minimum recommended exposure time per page (phase) for a three-line CMS is 3 s, aside from consideration of any particular set of driver characteristics. However, while some jurisdictions have selected briefer exposure times, the increasing number of older drivers on limited-access highways makes an even stronger case for the 3-s minimum per page. Reading time is the time it actually takes a driver to read a message sign. In instrumented vehicle studies conducted in light traffic with familiar drivers on a rural freeway, reading times averaged 1 to 1.5 s per unit of information (Mast and Ballas, 1976). Reading times under "loaded" driving conditions would be higher, such as under extreme geometry, heavy traffic volumes, large volumes of truck traffic, traffic conflicts, or poor climatological conditions. More recent field research using unfamiliar drivers has indicated that a minimum exposure time of 1 s per short word (four to eight characters) or 2 s per unit of information, whichever is larger, should be used (Carvell, Turner, and Dudek, 1978; Messer, Stockton, and Mounce, 1978; Weaver, Richards, Hatcher, and Dudek, 1978; Dudek, Huchingson, Williams, and Koppa, 1981). A unit of information is a data item given in a message that can answer one of the following questions: (1) What happened?; (2) Where?; (3) What is the effect on traffic?; (4) For whom is the advisory intended?; and (5) What driver action is advised? Thus, the exposure time for a three-line message could vary from 3 s to as much as 6 s, with each phase of a portable CMS at the lower end of this range and with each permanent CMS phase (page) at the upper end, due to differences in the number of characters per line. Reducing the exposure time per phase is warranted only when information is being repeated. For example, a three-line message may be displayed for only 2.5 s if it is a second phase of a two-phase message that repeats one or two lines from the first phase. If the second phase presents new information, the recommended *minimum* exposure time for both phases remains 3 s.

For a given operating speed, exposure will increase with increasing legibility distance. For example, an overhead sign message that is legible at 198 m (650 ft) will be exposed to drivers traveling at 88 km/h (55 mi/h) for approximately 8 s. With a legibility distance of 305 m (1000 ft), the message will be exposed for about 12 s. Legibility distances for a portable CMS vary from the minimum of 200 m (650 ft) specified by *MUTCD*, Part 6 and the American Traffic Safety Services Association (ATSSA) to more than 305 m (1000 ft), depending on the technology. A permanent CMS generally has a legibility distance in the higher range of 274–366 m (900–1200 ft). However, there is a point at which a sign becomes unreadable during a driver's approach to a CMS, which reduces the legibility distance, particularly for a side-mounted CMS. This unreadable distance, which is dependent on the number of lanes and the sign technology, as well as how far the sign is set back from the roadway edge or how high above the roadway it is mounted, ranges from 85 m to 128 m (280 ft to 420 ft). Therefore, in an existing system, required exposure times dictate the maximum length of a message that can be displayed, and *in all cases, it is desirable that motorists be able to read the entire message on a [unobstructed] CMS twice*.

The calculated maximum exposure duration of a message should not exceed 9 s. For two-phase messages, a separate requirement is needed to meet the needs of drivers. In this case, 3 s is added to the required exposure time because of the asynchrony between the time the driver can read the CMS and the onset of the CMS phase displayed. In other words, the phase that the driver reads initially may have already been displayed for 2 s by the time he or she can read it. Thus, the driver will not have enough time to read this phase and will need to view that phase again. The net result is that 3 s needs to be added to the required exposure time to allow drivers to read the phase that first came into view a second time. Since the maximum recommended exposure time is 9 s, only 6 s of actual message reading time is allowed on a two-phase CMS, whereas the full 9 s can be used for a single-phase message. The important point here is that single-phase messages can more efficiently convey information to drivers. When use of a single-phase CMS is not possible because of message length, multiple devices with a single phase on each device will be superior to multiple phases on a single device. Part 6 of the *MUTCD* (section 6F.52) refers the practitioner to table 6C-1 to determine separation distances between multiple portable CMS's placed on the same side of the roadway.

For these reasons, the *maximum* number of phases used to display a message on a permanent CMS should be two. The most effective format for CMS message presentation is a single phase that consists of a maximum of three units of information; however, if two phases are required, each should be worded so that it can stand alone and still be understood. Portable CMS devices, although limited to fewer characters per line, should also be restricted to two phases. At high speeds (88 km/h [55 mi/h]), a driver may only have 2.8 to 4.6 s to read a message on a side-mounted CMS, depending on the available legibility distance. For this reason, messages should be restricted to one phase at high speeds.

The motorist's need for rapid understanding and integration of message components also focuses attention on the formatting of multi-word text displays. The main concern is with "units of information"—i.e., where and how to divide phrases—and with the use of abbreviations and contractions in CMS messages. These formatting issues are discussed below.

Work zones constitute driving situations that require a large amount of controlled processing, and data show that cognitive ability scores that measure processing efficiency decline with age (Ackerman, 1987). In fact, sensory memory, working memory, and divided attention all show a decline with age and must be considered in the display of messages on CMS's. This reinforces the conclusion that a message should be limited to a single phase, or certainly no more than two, because multiple phases will interfere with message comprehension. There is also considerable evidence that older adults have poorer working memory function than younger adults (Salthouse, 1991; Salthouse and Babcock, 1991). This indicates that message length should be limited to the fewest, most relevant units possible. Finally, older adults are particularly disadvantaged when they are required to use working memory to manage multiple tasks (Ponds, Brouwer, and van Wolffelaar, 1988). Van Wolffelaar, Brouwer, and Rothengatter (1990) found that there is a disproportionately greater problem for older adults in divided-attention situations and they directly linked this to a higher crash rate for older adults in time-pressured, complex traffic situations.

The minimum required information for traffic management includes: (1) a statement of the problem; and (2) the action statement(s)—i.e., a driver needs to know what to do and one good reason for doing it. Additional elements are included as needed for a specific situation. The key here is not to burden the driver with unnecessary information. Only about two-thirds of the drivers are able to recall completely four pieces of information (problem, effect, attention, and action); however, 80 to 90 percent can recall the action message (Huchingson, Koppa, and Dudek, 1978). Two problems in message presentation must be avoided: (1) providing too much information in too short a time; and (2) providing ambiguous information that leaves either the intent of the message or the desired driver response uncertain.

The first problem does not refer solely to reading time difficulties, as discussed above; instead, it refers to the number of ideas, or "information units," contained in a message. Certainly, the number of words displayed on a sign is important, but so is the manner in which words are grouped. Units containing one word (DELAY), two words (DELAY AHEAD), or many words (MAJOR DELAY AT OAK STREET) are equally difficult to remember when the display is no longer in sight. However, a series of, say, six units of information in a message displayed on a permanent CMS will be easier to remember if presented in two phases of three units each rather than if all six units are presented on a single phase. Studies have concluded that no more than three units of information should be displayed on one sequence when all three units must be recalled by the drivers (Huchingson et al., 1978; Dudek et al., 1981; Gish, 1995).

Gish (1995) conducted a human factors laboratory study addressing the perceived timeliness, accuracy, and credibility of CMS messages using both younger (ages 16 to 33) and older (ages 65 to 84) test subjects. Results showed that correct recall of the first CMS phase (a downstream speed advisory) was nearly 100 percent for both age groups. However, successive phases of information (containing downstream delay and route diversion information) were recalled less accurately. For the delay information (second phase), correct recall for the younger subjects was about 82 percent, versus 60 percent for the older subjects. For route numbers (third phase), correct recall was 55 percent for the younger subjects and 19 percent for the older subjects. These results reinforce the earlier recommendation that a maximum of two phases should be used.

When a message *must* be divided into two phases, it is desirable to repeat key words from the first phase on the second phase, to ensure that all drivers see the message at least once. This also allows information rehearsal, as provided by an additional "learning trial," which will facilitate message recall when the device is no longer in sight. A recommended standard practice is therefore to put the problem on line 1, the location on line 2, and alternate either the effect and action *or* diversion information on line three, repeating lines 1 and 2 on both phases.

The second type of problem can occur when an unfamiliar word or abbreviation is used, when a word is hyphenated or a phrase is divided inappropriately, or when an abbreviation or a word can mean different things in different word pairings or contexts. Ambiguity occurs, for example, when CENTER LANE is used on a freeway with four or more lanes in one direction. Another example is the use of LANE CLOSED versus LANE BLOCKED to denote a prolonged closure for construction/maintenance versus a temporary blockage due to a crash or stall. To foster the most simple and consistent practice for motorists, LANE CLOSED is recommended under both roadwork and incident conditions, because at the time the message is displayed, the lane is effectively closed. Finally, neither FREEWAY BLOCKED nor FREEWAY CLOSED should ever be used when at least one lane is open to traffic.

Abbreviations also have the potential to be misunderstood by some percentage of drivers, exacerbating message comprehension problems for individuals with (age-related) diminished capabilities. It has been determined that certain abbreviations are understood by at least 85 percent of the driving public independent of the specific context (e.g., BLVD = boulevard). A second category of abbreviations are understood by at least 75 percent of the driving population, but *only* with a prompt word (e.g., LOC means "local" when shown with "traffic"). Other abbreviations are prone to be frequently confused with another word (e.g., WRNG could mean either "warning" or "wrong") and should be avoided. The following are lists of abbreviations in three categories, extracted from Dudek et al. (1981): (1) those that are acceptable (understood by at least 85 percent of the driving population) when shown alone (table 50); (2) those that are *not* acceptable and, therefore, should *not* be used (table 51); and (3) those that require a word to be used as a prompt (table 52). Table 50 also includes abbreviations taken from the *MUTCD*, as well as common contractions used in the English language. The abbreviations in these tables have been incorporated into the *MUTCD* (FHWA, 2000) as tables 1A-1 through 1A-3 in section 1A.14; section 6F.52 of the *MUTCD* states that when abbreviations are used on a CMS, they should be easily understood. Practitioners are referred to section 1A.14.

Table 50. "Acceptable" abbreviations for frequently used words.
Source: Dudek, Huchingson, Williams, and Koppa (1981).

Word	Abbreviation
Alternate	ALT
Avenue	AVE
Boulevard	BLVD
Can Not	CAN'T
Center	CNTR
Do Not	DON'T
Emergency	EMER
Entrance, Enter	ENT
Expressway	EXPWY
Freeway	FRWY, FWY
Highway	HWY
Information	INFO
It Is	IT'S
Junction	JCT
Left	LFT
Maintenance	MAINT
Normal	NORM
Parking	PKING
Road	RD
Service	SERV
Shoulder	SHLDR
Slippery	SLIP
Speed	SPD
Street	ST
Traffic	TRAF
Travelers	TRVLRS
Warning	WARN
Will Not	WON'T

Table 51. Abbreviations that are "not acceptable."
Source: Dudek, Huchingson, Williams, and Koppa (1981).

Abbreviation	Intended Word	Common Misinterpretation
ACC	Accident	Access (Road)
CLRS	Clears	Colors
DLY	Delay	Daily
FDR	Feeder	Federal
L	Left	Lane (Merge)
LT	Light (Traffic)	Left
PARK	Parking	Park
POLL	Pollution (Index)	Poll
RED	Reduce	Red
STAD	Stadium	Standard
WRNG	Warning	Wrong

Table 52. Abbreviations+ that are "acceptable with a prompt."
Source: Dudek, Huchingson, Williams, and Koppa (1981).

Word	Abbreviation	Prompt
Access	ACCS	Road
Ahead	AHD	Fog*
Blocked	BLKD	Lane*
Bridge	BRDG	[Name]*
Condition	**COND**	**Traffic***
Congested	**CONG**	**Traffic***
Construction	CONST	Ahead
Downtown	**DWNTN**	**Traffic***
Eastbound	**E-BND**	**Traffic**
Exit	EX, EXT	Next*
Express	EXP	Lane
Frontage	**FRNTG**	**Road**
Hazardous	HAZ	Driving
Interstate	I	[Number]
Local	**LOC**	**Traffic**
Major	MAJ	Accident
Mile	MI	[Number]*
Minor	MNR	Accident
Minute(s)	MIN	[Number]*
Northbound	**N-BND**	**Traffic**
Oversized	OVRSZ	Load
Prepare	PREP	To Stop
Pavement	PVMT	Wet*
Quality	QLTY	Air*
Roadwork	**RDWK**	**Ahead [Distance]**
Route	RT	Best*
Southbound	**S-BND**	**Traffic**
Temporary	**TEMP**	**Route**
Township	**TWNSHP**	**Limits**
Turnpike	TRNPK	[Name]*
Upper, Lower	UPR, LWR	Level
Vehicle	VEH	Stalled*
Westbound	**W-BND**	**Traffic**
Cardinal Directions	N, E, S, W	[Number]

+ The words and abbreviations shown in normal type are understood by at least 85 percent of the driving population. Those shown in boldface type are understood by at least 75 percent of the driving population, and public education is recommended prior to their usage.

* Prompt word should precede abbreviation.

C. Design Element: Channelization Practices (Path Guidance)

Table 53. Cross-references of related entries for channelization practices (path guidance).

Applications in Standard Reference Manuals	
MUTCD (2000)	*Traffic Engineering Handbook* (1999)
Sect. 6B.01 Sects. 6C-05, 6C.08, 6D.01, 6F.12, 6F.13, 6F.20 through 6F.24, 6F.30, 6F.55, & 6F.69 Sects. 6G.04 & 6G.06 Sect. 6G.09 Sect. 6G.10 Sects. 6G.11 through 6G.18 Figs. 6H-3, 6H-5 through 6H-7, 6H-10 through 6H-12, 6H-15, 6H-18, 6H-21 through 6H-34, 6H-36 through 6H-44, & 6H-46 plus associated notes for each figure.	P. 420 Para. 1 P. 434, Sect. on *Channelizing Lines* Pp. 441-443, Sect. on *Channelizing Traffic Control Devices*

Channelization systems include the use of cones, posts, tubular markers, barricades, panels, drums, amber-flashing and steady-burn lights, and standard and raised/recessed pavement markings. They are used to direct motorists into the open lanes and to guide them through the work area. They must provide a long detection distance and be highly conspicuous under both daytime and nighttime conditions. Using data collected by the police, it has been estimated that anywhere from 80 to 86 percent of the crashes in work zones can be attributed to driver error (Nemeth and Migletz, 1978; Hargroves and Martin, 1980). Hargroves and Martin (1980) found that crashes with fixed objects within a work zone account for a greater percentage of all work-zone crashes than other crash types, such as rear-end or sideswipe. Nemeth and Migletz (1978) found that nighttime crashes are concentrated in the taper area. Humphreys, Maulden, and Sullivan (1979) identified the most significant problems with channelization in work zones as: (1) failure to use, or hazardous use of, temporary concrete barriers; and (2) inadequate or inconsistent use of devices and methods in closing roadways and establishing lane-closure tapers.

Older drivers, like alcohol-impaired and fatigued drivers, show reduced sensitivity to contrast. Olson (1988) pointed out that the brightness of a traffic control device is the main factor in its attention-getting capability: In a visually complex environment, the brightness must be increased by a factor of 10 to achieve conspicuity equivalent to that found in a low-complexity environment. A major problem at night is the reduction in contrast sensitivity, which makes it difficult to see even large objects when they cannot be distinguished from their background. Older drivers also have difficulty processing information due to less effective scanning behavior and eye movements, diminished visual field size, difficulty in selective attention, and slower decision-making. Inconsistent use of drums and traffic cones to delineate the travel path may be a particular problem for older drivers, especially when applied in the presence of the remnants of old lane markings, because such inconsistency is confusing and older drivers (and inattentive drivers) are not able to react as quickly to conflicting traffic cues (National Transportation Safety Board, 1992). To compensate for their slower information-processing capabilities, their reduced visual capabilities, and their slower reaction time, older drivers often drive more slowly. Although driver age was not studied, Hargroves and Martin (1980) found that slow-moving vehicles were overrepresented in work-zone crashes. Older drivers also show a reduction in lane-keeping ability, which is further compromised when they are required to attend to other tasks in unfamiliar surroundings. Finally, steering ability may be adversely affected by physical problems such as arthritis.

McGee and Knapp (1979) performed an analytic study to develop a performance requirement/standard for the detection and recognition of retroreflective devices (cones, drums, panels, and barricades) used in work zones. The performance standard developed in this study, presented in terms of visibility requirements (i.e., the distance at which motorists should be able to detect and recognize the devices at night) and established using the principles of driver information needs and the requirement for decision sight distance, calls for a minimum visibility distance of 275 m (900 ft) when illuminated by the low beams of standard automobile headlights at night under normal atmospheric conditions.

Pain, McGee, and Knapp (1981) evaluated the effectiveness of traffic cones and tubular markers, vertical panels, drums, barricades, and steady-burn lights in laboratory studies, in controlled field studies, and at actual construction sites. A total of 254 subjects between the ages of 17 and 60+ participated; more than half of the subjects were between ages 21 and 40, and 7 percent of the subjects were age 60 or older. Overall, there were no major differences between the device categories during the day. At night, barricades, panels, drums, cones, and tubular markers were also equivalent when the optimized cone and tubular marker retroreflectorization was used (two bands of retroreflective material for cones and one band for tubular markers totaling 96,800 to 129,000 mm^2 (150 to 200 in^2), or roughly the amount provided by a 300- to 350-mm [12- to 14-in] collar) of retroreflective material with a specific intensity per unit area (SIA) of at least 250. However, tubular markers and cones with 150 mm (6 in) of collar resulted in diminished nighttime performance. The variables manipulated in the cone optimization study included (1) the amount of retroreflectorization (44,500, 89,000, 133,600, 178,100, and 222,600 mm^2 [69, 138, 207, 276, and 345 in^2]), corresponding to single bands measuring 150, 260, 350, 430, and 500 mm (6, 10, 14, 17, and 20 in) wide; (2) the number of bands of retroreflective material (1, 2, or 3); (3) three types of retroreflectorization plus one internally illuminated cone (polycarbonate Reflexite with an SIA of 2000 at an entrance angle of -4° and an observation angle of 0.1°, high-intensity sheeting with an SIA of 300 at an entrance angle of -4° and an observation angle of 0.1°, and polycarbonate Reflexite plus vinyl Reflexite); (4) color of retroreflectorization (white and yellow), three sizes (450, 700, and 900 mm [18, 28, and 36 in] tall); and (5) three device spacings (half, regular, and double speed limit). The variables manipulated in the tubular marker study included: (1) the amount of retroreflectorization (14, 28, 43, 57, and 71 percent of the area covered, corresponding to bands measuring 150, 300, 450, 600, and 950 mm [6, 12, 18, 24, and 38 in] wide); (2) the number of bands (1, 2, 4, 6, or 8); (3) the same retroreflectorization levels and colors as for the cone study, three sizes (450, 700, and 1050-mm [18, 28, and 42 in]) tall; and (4) the same device spacings as described for the cone study.

In comparing the meaning of chevrons versus stripes, Pain et al. (1981) found that diagonal, horizontal, and vertical stripes conveyed no consistent directional information; chevrons, though less easily detected than the stripe patterns, effectively and unambiguously indicated that a movement to the left or right was required. Since diagonal, horizontal, and vertical stripes conveyed no consistent direction information, Pain et al. (1981) concluded that there was no reason to have a diagonal stripe pattern for left and right "sidedness." They pointed out, however, that only one direction of diagonal should be allowed in an array so there is always a consistent pattern or image on devices.

In terms of device spacing, comparisons of regular speed-limit spacing (16.8 m [55 ft] in the test), half-spacing (8.4 m [27.5 ft]), and double-spacing (33.5 m [110 ft]) of Type I barricades and 200-mm x 600-mm (8-in x 24-in) panels showed that changes in spacing produced little impact on driver behavior. There were no significant speed or lateral placement differences between half, regular, and double speed-limit spacing during the day. At night, however, when devices were placed at half-spacing, they produced a speed reduction, apparently from the illusion that the motorist was going faster than he or she actually was. Devices placed at double-spacing tended not to perform as well as when they were placed at regular speed-limit spacing, as drivers made lane changes and detected arrays of traffic control devices sooner with shorter spacings. From these findings, Pain et al. (1981) recommended that: (1) all devices be placed at speed-limit spacing for most conditions and, in all cases, along the taper or transition section; (2) if there is no construction work or hazard in the closed lane for a substantial length, or traffic delays, the spacing can be increased to no more than twice the speed limit; and (3) shorter spacings may prove to be useful where speed reduction is desired.

Device-specific findings by Pain et al. (1981) are as follows:

- *Traffic cones*: (1) They perform as well as other devices during the day, with long detection distances and adequate lane-change distances. (2) Bigger is better: 900-mm (36-in) cones are more effective than 700-mm (28-in) cones; 700-mm (28-in) cones are better than 450-mm (18-in) cones (and 450-mm cones should not be used on high-speed facilities). (3) At night, 96,800 to 129,000 mm^2 (150 to 200 in^2), or roughly the amount in a 300- to 350-mm (12- to 14-in) collar of highly retroreflective material (with an SIA of at least 250), is needed for effectiveness. Even higher brightness materials enhance driver response characteristics and are preferable. (4) Under both daytime and nighttime conditions, the two-band configuration outperformed the three-band configuration, and both outperformed the one-band configuration; therefore, two bands of retroreflective material are preferable on cones.

- *Tubular Markers*: (1) During the day, 700-mm and 1050-mm (28-in and 42-in) tubular markers are as effective as cones; however, 450-mm (18-in) tubular markers are ineffective and are not recommended for lane closures or diversions on high-speed facilities. (2) At night, tubular markers with at least a 300-mm (12-in) highly retroreflective band are as effective as cones. (3) The one-band configuration outperformed the two- and three-band configurations for tubular markers

- *Vertical panels*: (1) Laboratory results showed that compared with the barricade, the vertical panel is more easily detectable. (2) Vertical panels are as effective (detectable) as Type I barricades, and vertical panels promote earlier lane-changing than do barricades. (3) The minimum width dimensions of the panel should be 300 mm (12 in) rather than 200 mm (8 in), especially when used at night and on high-speed facilities.

- *Drums*: (1) Drums are highly visible and detectable from long distances, during both day and night. (2) Drums promote lane-changing further upstream of the taper than other devices. (3) Drums are associated with a reduction in speed. (4) Drums are a dangerous object when hit.

- *Barricades*: (1) The Type I barricade is as effective as other devices. (2) The Type II barricade is no more detectable than the Type I barricade. (3) The 300-mm x 900-mm (12-in x 36-in) barricade is more conspicuous than the 200-mm x 600-mm (8-in x 24-in) barricade.

Other findings were reported for comparisons of steady-burn lights and Type II and Type III sheeting. The steady-burn lights provided the longest detection distances at night compared with all other materials, and they more than tripled the distance (or zone) in which lane-changing occurred before the taper. In comparisons of Type II sheeting and Type III sheeting on cone and tubular marker optimization tests, Type III was significantly better at night on a flat road. Narrow-angle sheeting, even though offering high brightness, was not effective under certain sight geometry characteristics, such as hills and curves. Type III sheeting and steady-burn lights were comparable in terms of point-of-lane-change and array detection distance; however, the authors noted that the effect of vertical or horizontal curvature must be considered.

There have been mixed results regarding the effectiveness of steady-burn lights in highway work zones. The use of steady-burn lights mounted on channelizing devices has been shown to significantly influence driver behavior in some work-zone configurations, and they are particularly effective in left-lane closures (KLD Associates, 1992). Although drivers age 55 and older consistently showed poorer performance than younger drivers in all study conditions, evidence was found that the use of lights improved the performance of older test subjects. The variables manipulated in this study included work-zone configuration (left-lane, right-lane, shoulder closures), device type (panels versus drums), and light placement (every device, alternate devices, no lights). Drivers of all ages were able to identify lane and shoulder closures from greater distances when lights were used on channelization devices, as opposed to when the channelizing devices were used alone. Steady-burn lights produced a higher percentage of correct responses (determining the direction toward which the channelizing devices were leading) for all driver age groups when used in left-lane closures than in right-lane closures. Interestingly, the use of lights on *every other* drum or vertical panel (placement on alternate devices) generated more correct responses than the use of lights on consecutive devices. More generally, the literature suggests that in environments characterized by high-speed operations, compromised visibility due to inclement weather, and/or complex maneuvers required as a result of work-zone configuration, the deployment of steady-burn lights should be considered on *all* channelizing devices used for right-lane closures.

However, Pant, Huang, and Krishnamurthy (1992) obtained a different result when they examined the lane-changing behavior of motorists in advance of tapered sections as they drove an instrumented vehicle through work zones during the day, at night when steady-burn lights were placed on drums, and at night when the steady-burn lights were removed. They measured the traffic volume at several locations in each lane in advance of the taper. Results showed that the steady-burn lights had little effect on driver behavior in the work zones studied. It was concluded by Pant et al. that steady-burn lights have little value in work zones that employ drums with high-intensity sheeting *and* a flashing arrow panel as channelizing devices.

Opiela and Knoblauch (1990) conducted laboratory and field studies to determine the optimal spacing and use of devices for channelization purposes in the taper or tangent sections of work zones. In the laboratory study, the recognition distances of eight different device types, spaced at the standard distance and at 1.5 and 2.0 times the standard distance, were measured for 240 subjects. Results indicated variability between the performance of most channelizing devices across the spacings tested. Right- and left-lane closures were then used at six actual work zones to test the various device spacings under both daytime and nighttime conditions. Field data were collected at four points equally spaced over 610 m (2000 ft) before the work zone and the activity at the start of the taper for the lane closure according to the premise that the most effective treatment would minimize the percentage of traffic in the closed lane at the start of the taper. Statistical analysis of 2,100 observation periods lasting 5 min each showed that neither type of device (round drums, oblong drums, Type II barricades, and cones with retroreflective collars) nor device spacings (16.8, 24.4, and 33.5 m [55, 80, and 110 ft]) had a significant effect on driver lane-changing behavior.

Cottrell (1981) also found that driver lane-change response was not strongly dependent on the channelizing device employed in a work-zone taper. The objective of this study was to evaluate the effectiveness of alternative orange-and-white chevron patterns on vertical panels and barricades that form an arrow pointing in the direction in which traffic is being diverted, compared with traffic cones, simulated drum vertical panels, and Type II barricades and vertical panels with standard orange-and-white striping patterns. The measure of effectiveness was the position of the lane-changing behavior relative to the transition taper. Although the subjective evaluation revealed that chevron patterns were preferred over the presently used patterns because of their clear directional message, the positions of the lane-changing behavior were similar for the stripes and chevrons. With respect to the spacing of devices, it was generally found that lane-changes occurred more frequently at greater distances from the taper when the devices were spaced every 12 m (40 ft), as opposed to every 24.4 m (80 ft).

In a supplemental test, the effectiveness of the concrete safety-shaped barrier (CSSB), also referred to as a "Jersey" barrier in some jurisdictions, was compared with that of the channelizing devices studied (Cottrell, 1981). The barrier was marked with steady-burn warning lights and 150-mm (6-in) reflectors and had a slope of 16:1 for the 58.5-m (192-ft) taper. The CSSB was rated to be equal to the cone during the day and lower than all other devices based on the lane-change positions. It was recommended that a supplemental taper of channelization devices be used with the CSSB. In a study of concrete barrier visibility, Pain et al. (1981) found that retroreflectors were superior to retroreflectorized tape. Logically, the most conspicuous types of retroreflective devices, such as those containing cube-corner lenses, will be potentially the most effective in this regard.

Overall, Pain et al. (1981) concluded that most devices show relatively successful detection and path-guidance performance. However, a major deterrent to effectiveness is not the device itself; instead, poor positioning, dirt, and overturned devices destroy the visual line or the path created by the channelizing devices. Therefore, *although use of appropriate devices is important, of equal importance is conscientious set-up and care of channelizing devices used in the work zones.*

In consideration of the threat posed to drivers by passenger compartment intrusion or interference with vehicle control or the threat to workers and other traffic from impact debris, plastic drums, cones, tubular markers, and vertical panels used as channelizing devices presented no hazard in full-scale vehicle crash tests (Bryden, 1990). However, Types I and II barricades and portable signs and supports formed impact debris, which was often thrown long distances through work zones, posing a threat to workers and other traffic. The American Traffic Safety Services Association (ATSSA) is opposed to the use of metal drums in work zones as channelizing devices, since they pose a hazard to motorists as well as workers in the zone (TranSafety, 1987). They suggest the use of plastic drums, which are safer. Riedel (1986) described studies showing that a substantial number of work-zone crashes occur in the taper and the crossover where channelization devices are located. The frequency of crashes involving drums has led to the use of forgiving devices such as plastic drums, which in tests have been shown to be safer than steel drums. Juergens (1972) noted that because barricades are inherently fixed-object hazards, they should not be used as primary delineation to guide traffic. Furthermore, they should not be used unless the construction hazard the motorist may encounter is greater than the hazard of striking the barricades. A concern with the use of steady-burn lights mounted on channelizing devices was highlighted in full-scale vehicle crash tests evaluating the performance of work-zone traffic control devices, where warning lights attached to these devices were thrown free, posing a potential threat to workers and other traffic (Bryden, 1990).

D. Design Element: Delineation of Crossovers/Alternate Travel Paths

Table 54. Cross-references of related entries for delineation of crossovers/alternative travel paths.

Applications in Standard Reference Manuals	
MUTCD (2000)	
Sects. 6F.68 through 6F.72	Sect. 6G.11
Sects. 6F.64, 6F.74, & 6F.75	Sect. 6G.13
Sects. 6F.61, 6F.62, & 6F.67	Sect. 6G.19
Sect. 6G.04	Figs. 6H-7, 6H-9, 6H-19, 6H-20, 6H-39 through 6H-41, & 6H-45
Sect. 6G.10	plus associated notes for each figure.

Studies have established that: (1) a substantial proportion of construction work-zone crashes occur in the taper and the crossover, where channelizing devices are usually located; (2) darkness is associated with an increase in the frequency of crashes in these areas; and (3) construction zones are associated with increases in the incidence of fixed-object, rear-end, and head-on crashes (Graham, Paulsen, and Glennon, 1977). Nemeth and Rathi (1983), studying crash types in construction zones on the Ohio Turnpike, found that 52.4 percent of the crashes were with fixed objects and 68.3 percent of the crossover crashes involved collisions with channelizing devices or other objects. In this study, 69.4 percent of the crashes at the first curve of a crossover occurred at night. Nemeth and Migletz (1978) found that 60.7 percent of single-vehicle fixed-object crashes were collisions with drums and 29.8 percent of all crashes involved collisions with drums. They also found that the proportion of crashes involving construction objects (drums) at night is significantly higher than the proportion of construction object crashes during the day. The results of these studies highlight the need for highly conspicuous and properly installed and maintained channelizing devices.

The relationships between the functional capabilities of older drivers and their performance that are likely to be of greatest operational significance as they approach and negotiate a crossover in a work zone can be summarized as follows: Age-related declines in acuity (both static and dynamic) and contrast sensitivity will delay recognition of channelizing devices and pavement markings and will delay comprehension of the information provided by advance warning signs. This information loss in the early stages of the driver's vehicle control task will be compounded by the attentional and decision-making deficits that are shown to increase with increasing age, with age differences in performance magnified as serial processing demands for conflict avoidance and compliance with traffic control messages increase during the approach to the work zone. Age-related decrements in the useful field of view, selective attention, divided attention, and attention-switching capabilities will slow the initiation of a driver's response when a lane change is required prior to the transition zone or when maneuvering through channelization across the median. In addition, less efficient working memory processes may translate into riskier operations for older drivers in unfamiliar areas if concurrent search for and recognition of navigational cues is required, since such demands disproportionately tax "spare capacity" for lane-keeping and conflict avoidance for older operators. Finally, the execution of vehicle-turning movements becomes more difficult for older drivers as bone and muscle mass decrease, joint flexibility is lost, and range of motion diminishes. Simple reaction time, while not significantly slower for older drivers responding to expected stimuli under nominal operating conditions, suffers operationally significant decrements with each additional response to an unexpected stimulus, e.g., as required in emergency situations. In addition, older drivers' increased sensitivity to glare and reduced adaptational ability to darkness will compound the difficulties described above while driving at night.

The National Transportation Safety Board (NTSB) has expressed concern about the lack of positive separation of opposing traffic in work zones (NTSB, 1992). The NTSB uses "positive barrier," or "positive separation of traffic," to refer to the use of concrete barriers to separate traffic. (A number of States distinguish between these terms, using "positive separation" to describe various channelization treatments that do not necessarily involve use of a physical concrete barrier.) The NTSB (1992) emphasizes that, *"Fatal crash rates increase significantly when an interstate highway is switched from a four-lane, divided operation to a two-lane, two-way operation (TLTWO) during construction work."* Research bearing on the use of channelization and barrier delineation for TLTWOs is described below.

A crossover requires a change in direction and may require a reduction in speed. This requires adequate advance warning of the lane and speed reduction, conspicuous and unambiguous delineation/channelization in the transition zone, and conspicuous separation of opposing traffic the length of the TLTWO. One survey of drivers in Houston, TX, and Dallas, TX, by Hawkins, Kacir, and Ogden (1992) found that only half of the respondents correctly understood that they should turn before reaching the CROSSOVER sign (D13-1) when this device was shown in a field placement in an arterial work zone. Of course, the D13-1 sign panel is identified in the *MUTCD* as a device used in permanent installations on divided highways, not as a temporary device for use in construction zones. The poor comprehension by motorists of such an explicit message is alarming, nevertheless, and suggests the need for heightened conspicuity of guidance information in this situation. Hawkins et al. recommended that the spacing of channelizing devices be decreased in the vicinity of a crossover to reduce drivers' confusion.

Next, Pang and Yu (1981) conducted a study to verify whether concrete barriers were justified at transition zones adjacent to TLTWOs on normally divided highways, based on crash experience in several construction-zone TLTWOs. They found that 34 of the 44 crashes that occurred in TLTWOs were within the transition zone. Four head-on crashes occurred on TLTWO segments away from the transitions. The transition zone was defined as the roadway section at which traffic flow was converted from a four- to a two-lane operation and vice versa. The absence of opposing traffic precluded the occurrence of head-on crashes during the study period; however, more than one-half of the crashes (56 percent) had the potential of becoming head-on collisions. The authors concluded that on relatively low-volume highways, delineation devices appear to be adequate at transition zones, assuming they are placed properly. A regression analysis provided by Pang (1979) indicated that as annual average daily traffic increases, the crash rate at transition zones also increases, with a concurrent increase in the head-on crash rate at the transition zone.

Project duration and approach speed are two other variables that appear to affect the head-on crash rate at transitions (Pang and Yu, 1981). Graham (1977) concluded that as project duration increases, the crash rate at the transitions decreases. Expectancy issues were highlighted as a plausible explanation. Pang and Yu (1981) reported that because the crash rate in the transition zone increases with shorter project duration, concrete barriers may be necessary for short-term projects. However, long-term projects are expected to have a greater number of crashes owing to a longer period of exposure. Thus, installation of concrete barriers would be more economically justified for long-term projects than for short-term ones. With regard to approach speed, it can be expected that as speed to the transition increases, the chances of a head-on collision would also increase, due to the tendency of vehicles to stray out of their lanes at curves such as those present in transition zones. Pang and Yu (1981) suggested that concrete barriers appear to be justified at transition zones where approach speeds are high.

The conspicuity of concrete safety-shaped barriers (CSSBs) is an important issue. Their composition provides little contrast with the roadway pavement, making them difficult to see at night, particularly in the rain, and under opposing headlight glare conditions. Proper barrier delineation treatments will provide drivers with a defined path during darkness and adverse weather conditions. Standard barrier delineation treatments include Type C steady-burn warning lights on top of the barrier, retroreflective devices on the top or side of the barrier, vertical panels placed on top of the temporary concrete barrier, and retroreflective pavement markings on the side of the barrier. The results of studies of barrier delineation in work zones have been mixed (Ullman and Dudek, 1988). For instance, Mullowney (1978) suggested that delineation should be mounted on the top of the barrier so it will retain its reflectivity longer and require less maintenance. However, Ogwoaba (1986) recommended side-mounted concrete barrier delineation so that the delineators are not masked by oncoming headlight glare. The size and brightness of the delineators is another controversial topic, with some studies suggesting the use of larger but less bright devices (Davis, 1983; Bracket, Stuart, Woods, and Ross, 1984; Kahn, 1985) and others recommending smaller, brighter reflectors (Mullowney, 1978; Ogwoaba, 1986). Kahn (1985) found that the delineation of portable concrete barriers improved considerably through the use of cylindrical reflectors on top and smaller units on the side of the barrier at 7.6-m (25-ft) intervals. Delineator spacings ranging from 7.6 m to 61 m (25 ft to 200 ft) have been recommended by various studies.

Ullman and Dudek (1988) conducted a study of five barrier delineation treatments, using observations of driver performance to determine how different delineator types, spacings, and mounting positions on the barrier affect nighttime traffic operating in the travel lane next to the barrier. An additional objective of the study was to determine how the visibility and brightness of different types of delineators deteriorate over time because of dirt and road film; in a controlled field study, drivers ages 18 to 56 were asked to provide subjective evaluations of delineator brightness. The study was not conducted at a work zone, but was conducted on an illuminated urban freeway with four lanes in each direction. The CSSB was located 0.3 m (1 ft) from the inside travel lane. The five delineation treatments were: (1) top-mounted cube-corner lenses at 61-m (200-ft) spacing; (2) side-mounted cube-corner lenses at 15.2-m (50-ft) spacing; (3) top-mounted retroreflective brackets at 15.2-m (50-ft) spacing; (4) side-mounted retroreflective brackets at 61-m (200-ft) spacing; and (5) top-mounted retroreflective cylinders at 15.2-m (50-ft) spacing. The cube-corner reflector (treatments 1 and 2) had a diameter of 81 mm (3.25 in). The brackets (treatments 3 and 4) were 75 mm (3 in) high and 106 mm (4.25 in) wide, and were covered with high-intensity sheeting. The cylindrical tube (treatment 5) had a diameter of 75 mm (3 in) and was 150 mm (6 in) high, and was wrapped with high-intensity sheeting. Before-and-after data were obtained for the following measures of effectiveness: lane distribution, lane straddling, and lateral distance from the left rear tire to the bottom of the CSSB.

Results of the driver performance data collected by Ullman and Dudek (1988) showed that the treatments had very little practical effect on lane distribution. Lane-straddling rates at all of the treatment segments were low during the higher volume nighttime hours; however, a significant increase in lane straddling occurred for treatment 2. The data suggested that the combination of close delineator spacing and the side-mounted position may make some drivers too apprehensive of driving near the barrier. Lateral distance data showed significant differences during the higher volume nighttime hours for treatment 4 and treatment 5. Lateral distance distributions shifted away from the barrier at treatment 4 and closer to the barrier at treatment 5. Subjective evaluations for clean delineators showed brightness rankings to be the same for all treatments. Treatments 1 through 4 received adequate ratings from at least 80 percent of the subjects, while treatment 5 was rated as adequate by only 50 percent of the subjects. With respect to each treatment's relative effectiveness in helping drivers maintain a safe travel path next to the CSSB, the rankings did not differ significantly; however, treatment 5 again received the worst score. Subjects stated that side-mounted delineators were preferable to top-mounted delineators because side-mounted delineation provided a more direct line of sight, a better indication of the location of the wall, and a more realistic perception of the lane width. For dirt-covered delineators, treatment 2 was rated as being the brightest and the most effective, while treatment 5 was rated as being the dimmest and the least effective. Although further research was deemed necessary, the study authors recommended the use of cube-corner lenses for delineating CSSBs in narrow freeway median applications, because these delineators do not lose their reflectivity due to dirt and grime as quickly as those covered with high-intensity sheeting. In addition, for situations with limited lateral clearance, as is common with TLTWOs, top-mounted delineation is recommended, because side-mounted close delineator spacing results in lane straddling if the barrier is located close to the travel lanes. Although subjects indicated a preference for close spacings, driver performance data did not show any differences between 15.2-m (50-ft) and 61-m (200-ft) spacing. The authors recommended that a 61-m (200-ft) spacing be considered a maximum and that closer spacings may be necessary for CSSBs on sharp curves. The recommendations were also deemed appropriate for CSSBs in work zones.

On divided highways with narrow medians, which are often created when barriers are used in crossover situations in work zones, drivers are subject to blinding glare from opposing vehicle headlights. This is particularly problematic for older drivers who have a reduction in their ability to adapt to darkness and an increased sensitivity to glare. This results in reduced visibility of roadway alignment and channelization, and increases the possibility of crashes. Glare screens can solve the problem, as well as reduce rubbernecking and its associated problems. The Pennsylvania Department of Transportation discontinued the use of the standard glare-control mesh screen in 1976, based on maintenance difficulties, and has employed a paddle-type system in its place (Maurer, 1984). The system consists of plastic airfoil-shaped paddles, which when mounted resemble a picket fence. The results of a 5-year study have shown that the paddle-type system reduces headlight glare satisfactorily and is more cost-effective, both in terms of installation and maintenance, than metal mesh screen. The system was also found to be beneficial as a temporary control for channelizing traffic around a construction work zone when screening was placed at the transition or the taper zone at the ends of the work zone (Maurer, 1984). Kelly and Bryden (1983) reported that a glare screen consisting of individual plastic louvers 900 mm (36 in) high, mounted vertically on a guiderail or median barrier spaced at 600-mm (24-in) centers, performed as expected in two safety improvement projects.

E. Design Element: Temporary Pavement Markings

Table 55. Cross-references of related entries for temporary pavement markings.

Applications in Standard Reference Manuals	
MUTCD (2000)	*Traffic Engineering Handbook* (1999)
Sects. 6F.55 through 6F.57 Sects. 6F.65 through 6F.67 Sect. 6G.05 Figs. 6H-7, 6H-12, 6H-14, 6H-24, 6H-29, 6H-32 through 6H-34, 6H-36, 6H-38 through 6H-42, & 6H-44 plus associated notes for each fig.	P. 440, Sect. on *Raised Pavement Marker* P. 442, Sects. on *Cones and Tubular Devices & Drums*

Preconstruction centerlines and edgelines that are not obliterated may confuse drivers about the exact locations of lanes. The National Transportation Safety Board (1992) has reported that although guidelines exist for proper signing and striping in construction areas, the traffic control techniques used in many jurisdictions are not in compliance with the guidelines. Lewis (1985) stated that if drivers are presented with conflicting information (as may be the case in a work zone), they will generally choose to follow the pavement, as the pavement itself is a primary source of information for drivers. This points to a need for unambiguous pavement delineation patterns in work zones to provide clear guidance—particularly at night and under adverse weather conditions—and to accommodate drivers with visual limitations, such as those associated with normal aging.

The research findings that have the greatest bearing on age differences in drivers' ability to acquire and use information provided by roadway delineation are a decline in spatial contrast sensitivity and acuity for older drivers, and a general slowing of responses because of deficits in visual search ability that slows discrimination of more important from less important information in a driving scene.

Discrimination of the boundaries of the traveled way often involves only slight differences in the brightness of the road surface versus the shoulder or surrounding land. The ability to obtain such "edge information" depends upon a driver's sensitivity to contrast. Age differences in contrast sensitivity, beginning at approximately age 40 and becoming progressively more exaggerated with advancing age, demonstrate significant decrements in performance for older persons (Owsley, Sekuler, and Siemsen, 1983). Under constant viewing conditions, older observers have lower contrast sensitivity, especially in situations where there is a reduction in ambient light levels. A 60-year-old driver requires 2.5 times the contrast needed by a 23-year-old driver (Blackwell and Blackwell, 1971).

Age decrements in visual search and scanning capabilities are widely reported in gerontological research. Rackoff and Mourant (1979) measured visual search patterns for 10 young (ages 21-29) and 13 older (ages 60-70) subjects as they drove on a freeway under daytime and nighttime conditions in low to moderate traffic. They reported that differences between young and older test subjects' performance were most apparent at night, and that older subjects required more time to acquire the minimum information needed for vehicle control. Thus, older drivers require delineation information that is optimal from the standpoints of both attention conspicuity and search conspicuity downstream, and that provides unambiguous path guidance cues for moment-to-moment steering control. Uncertainty about roadway heading and lane position has been cited by older driver focus group members as reasons for driving more slowly, for erratic maneuvers caused by last-second steering corrections, and for simply avoiding nighttime operations (Staplin, Lococo, and Sim, 1990). An exaggeration of the difficulties older drivers have in rapidly discerning the correct travel path may be expected in construction zones, where drivers must respond to temporary pavement markings that are often in competition with preexisting stripes and/or misleading informal cues provided by variation in the surface characteristics of the road, shoulder, or median.

These diminished capabilities must be considered in relation to specific information needs when negotiating work zones, while also taking into account the time (distance) in which these needs must be satisfied. The information needs may be loosely contrasted according to the discrimination of continuous versus discrete roadway features, i.e., the perception and recognition of the boundaries of the traveled way, as opposed to a singular location that must be avoided (e.g., an island, barrier, or abutment) or where a path selection decision must be acted upon (e.g., a ramp gore, pavement-width transition point, or intersection). Furthermore, delineation must provide information to a driver that permits roadway feature recognition both at long preview distances up to and sometimes exceeding 5 s of travel time and at the more immediate proximities (within 1 s of travel time) where attention is directed for instant-to-instant vehicle control responses.

An investigation of age-related differences in the required contrast for pavement delineation showed that an older driver (ages 65–80) test sample required a level of contrast 20 to 30 percent higher than a young/middle-aged (ages 19–49) comparison group (Staplin et al., 1990). The differences became exaggerated with glare as an independent variable. An inevitable consequence of these age differences is an increased reliance on delineation elements for path guidance by older drivers under nighttime conditions, especially against oncoming glare. The long preview time, as well as the instant-to-instant steering control cues provided by pavement markings, are critical to older drivers under these circumstances.

Raised pavement markers (RPMs) used for delineation of the centerline and edgelines in construction zones have been found to provide improved wet weather and nighttime reflectivity, and are particularly useful when lanes are diverted from their original path (Spencer, 1978). Davis (1983) reported that, compared with conventional pavement markings (e.g., paint), day-night/wet-night visible RPMs improved construction zone traffic performance significantly. In this study, the markers were associated with decreased lane-change frequency and nighttime lane encroachments. In before-and-after comparisons of crash frequencies in two construction projects, the number of crashes and fatalities decreased as a function of RPM installation (Niessner, 1978). In a study investigating vehicle guidance through work zones, Shepard (1989) recommended that closely spaced RPMs should be used as a supplement to existing pavement striping in areas where the roadway alignment changes.

Dudek, Huchingson, and Woods (1986) conducted a study on a test track to examine the effectiveness of temporary pavement markings for use in work zones. Ten candidate treatments were tested during the day, and the most effective treatments were examined at night. All treatments were tested only under dry weather/dry road conditions. The candidate treatments are presented in table 56 and included patterns with stripes, RPMs, and combinations of stripes and RPMs. Treatment 1 was the control condition in the study.

Table 56. Temporary pavement marking treatments evaluated by Dudek, Huchingson, and Woods (1986).

Treatment	Description
1*	4-ft stripes (4 in wide) with 36-ft gaps (control condition)
2*	2-ft stripes (4 in wide) with 38-ft gaps
3*	8-ft stripes (4 in wide) with 32-ft gaps
4*	2-ft stripes (4 in wide) with 18-ft gaps
5*	Four non-reflective RPMs at 3-1/3-ft intervals with 30-ft gaps and one retroreflective marker centered in alternate gaps at 80-ft intervals
6*	Three non-retroreflective and one retroreflective RPM at 3-1/3-ft intervals with 30-ft gaps
7	2-ft stripes (4 in wide) with 48-ft gaps
8	Treatment 2 plus RPMs at 80-ft intervals
9*	Two non-retroreflective RPMs at 4-ft intervals with 36-ft gaps plus one retroreflective RPM centered in each 36-ft gap
10	1-ft stripes (4 in wide) with 19-ft gaps

* Treatments evaluated both during the day and at night.

1 ft = 0.305 m
1 in = 25 mm

Results of both daytime and nighttime testing indicated that there were no practical differences between treatments when comparing measures of effectiveness developed from speed and distance measurements. Practical differences were arbitrarily defined as at least 6.5 km/h (4 mi/h) for speed measures and 0.3 m (1 ft) for distance measures. The greatest number of erratic maneuvers during the day occurred for treatments 7 and 8, which consisted of 0.6-m (2-ft) stripes and long gaps. Drivers referred to 0.6-m (2-ft) stripes as dots. The subjective data indicated that treatments 5, 6, and 9 were preferred, under both daytime and nighttime conditions. The reasons given were that RPMs clearly identify curves, are highly visible at a great distance, provide noise and vibration when drivers cross them, and stand out more than tape markings. Of the treatments without RPMs, treatment 3 was the drivers' choice for both lighting conditions, while treatment 2 was rated as being the least effective.

It should be noted that for temporary that pavement markings, the *MUTCD* specifies in section 6F.66 that the same cycle length as for permanent markings should be used (9 m [30 ft]), with markings at least 0.6 m (2 ft) long, and that half-cycle lengths with a minimum of 0.6-m (2-ft) stripes may be used for roadways with severe curvature.

Because subjects tend to perform best when in a controlled test-track setting, and because the range of performance measures are not always sensitive enough to discern small differences between candidate treatments, Dudek, Huchingson, Creasey, and Pendleton (1988) conducted field studies to compare the safety and operational effectiveness of 0.3-m (1-ft), 0.6-m (2-ft), and 1.2-m (4-ft) temporary broken-line pavement markings on 12.2-m (40-ft) centers in work zones. The study was conducted at night on rural TLTWO highways with 2.0-degree horizontal curvatures, level to rolling terrain, and average speeds between 80.5 km/h (50 mi/h) and 99.8 km/h (62 mi/h). In terms of speed, lateral distance, encroachment, erratic maneuver, and speed profile data for the sample of vehicles with headways of 4 s or more, there were no differences in driver performance between the 0.3-m (1-ft), 0.6-m (2-ft), and 1.2-m (4-ft) striping patterns. Analysis of subjective evaluations of the effectiveness of the markings found that the 0.3-m (1-ft) stripe was generally rated as being the poorest, but its mean ranking was not significantly different from that of the 0.6-m (2-ft) and 1.2-m (4-ft) stripes. Drivers generally preferred the longer stripes, but there was no evidence that the 0.6-m (2-ft) or 1.2-m (4-ft) stripes were superior to the 0.3-m (1-ft) stripe.

In a discussion of the conditions present during this research, Ward (1988) stated that all sites had 3.7-m (12-ft) lanes with 1.2-m (4-ft) to 3-m (10-ft) shoulders, the marking material was highly retroreflective yellow tape laid over very black new pavement overlays, and there were no edgelines; therefore, the drivers' focus was a "brilliant ribbon of yellow to follow," resulting in no difference in driver performance between the three stripe lengths. Most important was that none of the treatments were judged as extremely effective, although the 0.3-m (1-ft) stripe was rated as being the poorest, and there was a slight preference for the 1.2-m (4-ft) lengths. This is consistent with results obtained by Dudek et al. (1986), where subjects rated 2.4-m (8-ft) stripes with 9.8-m (32-ft) gaps as being the best striping treatment (when RPMs were not available). In the Dudek et al. (1986) study, drivers preferred the treatments with longer stripes, shorter gaps, and RPMs. Hence, the results of the Dudek et al. (1988) study may be applicable only to pavement overlay projects on TLTWO rural roadways, and may not translate to other highway work-zone situations.

Harkey, Mera, and Byington (1992) conducted a study to determine the effects of short-term (interim) pavement markings on driver performance under daytime, nighttime, and wet and dry weather conditions. The three marking patterns tested included: (1) 0.6-m (2-ft) stripes with 11.6-m (38-ft) gaps and no edgelines; (2) 1.2-m (4-ft) stripes with 11-m (36-ft) gaps and no edgelines; and (3) 3-m (10-ft) stripes with 9.2-m (30-ft) gaps and edgelines. The measures of effectiveness included lateral placement of the vehicle on the roadway, vehicle speed, number of edgeline and lane-line encroachments, and number of erratic maneuvers (e.g., sudden speed or directional changes and brake applications). For each operational measure, the 3-m (10-ft) markings resulted in better driver performance than either the 0.6-m (2-ft) or 1.2-m (4-ft) temporary marking patterns. Drivers traveled 1.2 km/h (0.76 mi/h) slower on segments with 1.2-m (4-ft) markings and 3.3 km/h (2.02 mi/h) slower on segments marked with 0.6-m (2-ft) markings than on segments marked with 3-m (10-ft) stripes and edgelines. In addition, compared with the 3-m (10-ft) pattern, drivers encroached over the lane-line or edgeline 66 percent more often in the presence of the 1.2-m (4-ft) temporary markings and 139 percent more often in the presence of the 0.6-m (2-ft) markings. These values increased dramatically under nighttime and wet weather conditions. Comparisons of driver performance between the 1.2-m (4-ft) and 0.6-m (2-ft) markings showed the following: (1) the speed at which drivers traveled decreased as the length of the lane line decreased; (2) drivers positioned their vehicles closer to the center of the lane as the length of the line increased; (3) the variability of vehicle placement within the lane increased as the length of the lane line decreased; (4) the number of encroachments increased as the length of the lane line decreased; and (5) all operational measures were negatively affected by adverse weather conditions. Results provided evidence of significant decreases in driver performance associated with both the 0.6-m (2-ft) and the 1.2-m (4-ft) markings, but drivers performed better with the 1.2-m (4-ft) stripes compared to the 0.6-m (2-ft) stripes. The results suggested that while it may not be practical to place full markings (3.0-m [10-ft]) segments with 9.0-m [30-ft] gaps as specified by *MUTCD* Part 3A.06) on a temporary basis, measures should be taken to prevent the reductions in driver performance that result in increased crash potential. Such measures include the use of longer temporary markings, the addition of RPMs for improved performance under adverse weather conditions, and the appropriate use of warning signs to indicate a change in the pavement marking pattern.

V. HIGHWAY-RAIL GRADE CROSSINGS (PASSIVE)

The following discussion presents the rationale and supporting evidence for Handbook recommendations pertaining to passive crossing control devices.

Design Element: Passive Crossing Control Devices

Table 57. Cross-references of related entries for passive crossing control devices.

Applications in Standard Reference Manuals				
MUTCD (2000)	AASHTO *Green Book* (1994)	*Roadway Lighting Handbook* (1978)	*Railroad-Highway Grade Crossing Handbook* (1986)	*Traf. Eng. Handbook* (1999)
Sect. 1A.13, *Highway-Rail Grade Crossing* Sects. 8A.01 & 8A.03 Figs. 8A-1 & 8A-2 Sects. 8B.02 through 8B.08 Fig. 8B-1 Sects. 8B.10 through 8B.14, 8B.16, & 8B.17 Fig. 8B-2 Sects. 8C.01, 10C.03 through 10C.05, & 10C.07 through 10C.14	Pp. 426-427, Sect. on *Railroad Grade Crossings* P. 440, Sect. on *Railroad Grade Crossings* Pp. 469-470, Sect. on *Railroad-Road Grade Crossing* P. 511, Sect. on *Railroad Crossings* Pp. 795-802, Sect. on *General*	Pp. 55-56, Sect. on *Control of Distribution Above Maximum Candlepower*	P. 10, Para. 3 P. 11, Para. 4 P. 12, Para. 2 P. 29, Para. 3 P. 30, Table 8 P. 31, Paras. 5-6 P. 33, Paras. 2 & 4 P. 53, Fig. 7 P. 57-58, Figs. 9-10 Pp. 66-69, Sects. on *New Hampshire Index & NCHRP 50* P. 71, Tables 19-20 P. 78, Para. 2 Pp. 80-81, Para. 6 & Fig. 15 P. 83, Para. 5 P. 84, Para. 5, 4th bullet P. 87, Para. 4, Last bullet Pp. 96-103, Sect. on *Passive Traffic Control Devices* Pp. 140-143, Sects. on *Illumination & Miscellaneous Improvements* P.173-176, Item 1 of each economic type analysis Pp. 177-180, Sect. on *Resource Allocation Procedure* P. 188, Para. 4 Pp. 196-199, Part of Sect. on *Traffic Control Devices* Pp. 201-202, Figs. 100-103 P. 206, Paras. 2-3 P. 215, Para. 4 P. 217, Para. 5 P. 219, Para. 4, 2nd bullet P. 220, Sect. on *Improved Signing* P. 226, Paras. 4 & 8 P. 227, Para. 1	Pp. 235-236, Sect. on *Yield Control* Pp. 242-243, Sect. on *Railroad-Highway Intersection Control* P. 414, Item 3 Pg. 423, Sect. on *Passive Signs* P. 435, Sect. on *Railroad Crossings Pavement Markings* Pp. 437-438, Sect. on *Delineators* P. 445, Sect. on *YIELD Sign Warrants*

For the approximately two-thirds of highway-rail grade crossings that are controlled by passive devices, "recognition errors" have been cited as the most frequent error type, accounting for 77 to 85 percent of the errors at crossbuck-only crossings (Berg, Knoblauch, and Hueke, 1982). This category of driver error was defined broadly by the study authors as "a breakdown in the detection and/or perception of the necessary information to negotiate the crossing safely." Given the vast body of evidence that sensory and perceptual capabilities decline as a function of age, it is reasonable to assert that the tasks of detecting an approaching train and judging its distance and speed pose exaggerated difficulty for older drivers. Older drivers are also slower than their younger counterparts in processing information (Berg et al., 1982), which was cited as a significant contributing factor by these researchers in decision errors—i.e., the "breakdown in the analysis of information or an incorrect choice of action"—at rail crossings by elderly and inexperienced drivers. Fambro (1999) similarly underscores these problems that disproportionately affect older road users in his comprehensive literature review, while summarizing a large body of research directed at engineering countermeasures to improve driver understanding of and behavior at grade crossings.

Studies in this area have examined how changes in signing, delineation, the use of rumble strips, and the introduction of nighttime illumination can variously affect drivers' allocation of attention; looking behaviors; braking/deceleration during an approach to a highway-rail grade crossing; and maneuver decisions to negotiate the crossing, as well as the actual vehicle-train crash experience at a site.

To begin, simply detecting the presence of a passive crossing, where the need to slow and look for a train is indicated, can be problematic, especially at night. Fambro (1999) cites the work of Russell and Konz (1980) who found that placement of the crossbuck sign (R15-1) does not make the best use of the vehicle's headlight beam pattern, which is aimed to the right and down. They state that certain combinations of headlight angle and roadway geometry at the crossing may result in a crossbuck sign luminance of zero. The *MUTCD* states that the sign height above the ground (2.8 m [9 ft]) may be varied as required by local conditions. Russell and Konz (1980) recommend lowering the crossbuck by 0.61 m (2 ft), which would increase the illuminance by 50 percent at 45.7 m (150 ft) and 69 percent at 76.2 m (250 ft). More recently, Russell and Rys (1994) conducted a small field study that confirmed that from a distance of 107 m (350 ft), a crossbuck at a height of 2.8 m (9 ft) measured to the center and 1.8 m (6 ft) from the right pavement edge received 8.4 percent of the vehicle headlight illuminance, compared to 14.2 percent if the crossbuck were lowered to 2 m (7 ft) above ground level, and 44 percent if the crossbuck were lowered to 0.9 m (3 ft) above the ground. Russell and Rys conclude that placement of the crossbuck as low as possible and using retroreflective tape on the full length of the crossbuck posts would make the best use of the greater headlight illuminance levels present near the ground.

Related work by Russell and Kent (1993) indicates that the conspicuity of passive grade crossings may be affected by adopting more effective delineation practices. These researchers conducted a before-and-after study to evaluate the effectiveness of five low-cost passive warning systems implemented at six different sites. The warning systems included combinations of conventional signing (YIELD) and an enhanced delineation treatment; novel signing (the "Buckeye Crossbuck," also termed the "Conrail Shield," as shown in figure 22, on p. 288 of this Handbook) plus an enhanced delineation treatment; two systems utilizing signing alone; and the enhanced delineation treatment alone. The enhanced delineation treatment included Type VII (ASTM D4956-01) retroreflective tape that provides for high brightness on straight approaches, as well as at wide observation angles (with SIA values ranging between 800 and 1000), placed on both sides of both crossbuck posts, and Type II, flexible roadside retroreflective delineators with high-intensity sheeting placed on the right side of each approach, spaced 15.1 m (50 ft) apart from the advance warning sign to the crossbuck post (a total distance of 366 m [1200 ft]) and extending an equal distance beyond the crossbuck post.

The measures of effectiveness in the Russell and Kent (1993) study included: approach speeds; brake-light activations; and head movement toward the tracks. One "before" study was conducted prior to installation of the new treatments, and two "after" studies were conducted, one at 2 months and the other at 7 months after treatment installation. Each observation period lasted for 3 to 4 days, from 2:30 p.m until 5:30 p.m., and from 9:00 p.m. until 12 midnight. Results indicated that only the locations where the delineation system alone (roadside delineators and high-brightness tape on both sides of the crossbuck posts) was installed showed statistically significant, long-term positive changes in more than one variable (deceleration rates and looking behavior). The researchers indicated that the added retroreflective devices and tape on the crossbuck posts provide high values of reflected light at all distances, especially under low-beam headlight conditions, concluding that the delineation treatment showed the most permanent improvement in driver behavior of the low-cost systems tested, and would be effective at grade crossings on highways in rural areas, particularly at night.

Brich (1995) similarly concluded that the use of very-high-intensity retroreflective sheeting (e.g., ASTM D4956-01 Type IX) applied to both the front and back of the crossbuck posts for the entire length of the post, plus double-sided crossbucks (blades) also with very-high-intensity retroreflective sheeting resulted in superior conspicuity of the crossing, compared to systems that contained double-sided crossbucks with Type IX sheeting, plus Type IX sheeting on only the back of the posts (full length); or double-sided crossbucks with Type IX sheeting, plus Type IX sheeting placed on the back of the posts in a 1.2-m-(4-ft-) long strip; or single-sided crossbucks with a strip of Type IX sheeting on the back of the crossbuck blades and also on the front and back of the posts (full length); or single-sided crossbucks with a strip of Type VII sheeting on the back of the crossbuck blades and also on the back of the posts (applied 0.91 m [3 ft] above ground level to the center of the crossbuck blades). Both the Type VII and IX sheeting materials provide high brightness at wide observation angles, but Type IX is designed to provide high brightness at shorter viewing distances (in the 91- to 183-m [300- to 600-ft] range). To delineate the crossbuck posts, a strip of 50-mm-(2-in-) wide sheeting was applied to a 75-mm-(3-in-) wide, 2.74-m-(9-ft-) tall aluminum strip that was mounted on the front and the back of the crossbuck post, or to the back of the post only, depending on the treatment being evaluated. The study by Brich (1995) was conducted in a laboratory using videotaped images of varying treatments filmed under low- and high-beam headlight illumination at night, during the approach to rail grade crossings. A train was also filmed traveling through each crossing.

Comments obtained from the subjects in the Brich (1995) study indicated that marking the full length of the crossbuck posts on the front and the back results in: (1) visually stabilizing the crossbuck and tying it to the ground, which provides a valuable reference point; and (2) making the part of the far-side post below the undercarriage of a moving train in the crossing visible, causing a flickering effect that alerts a motorist of the presence of a train. It was concluded that using double-sided crossbucks and marking the full length of both sides of both posts increases the visibility of the crossing; increases driver depth perception of the crossing; and increases the ability of a driver to detect a train in the crossing. The cost of a double-sided crossbuck with Type IX sheeting in 1995 was $87.50 and the cost of the sheeting and aluminum to be applied to the post was $18.74 per crossbuck assembly. Since most crossings use two crossbuck assemblies, the cost per crossing (crossbuck, post, aluminum strip, and retroreflective material) was $212.48.

The *MUTCD* (FHWA, 2000) includes the requirement to use a strip of retroreflective sheeting that is at least 50-mm-(2 in) wide on the front *and* back of each crossbuck support post for the full length of the post. In addition, it states that a strip of retroreflective sheeting shall be applied to the back of the crossbuck blades, except in locations where crossbuck signs are installed back-to-back. This updates the prior edition of the *MUTCD* (FHWA, 1988), which required only that the crossbuck sign itself be retroreflectorized.

Older drivers, with their decreased contrast sensitivity and need for increasing levels of light for night-driving tasks, would be expected to benefit disproportionately from increasing the detectability and conspicuity of railroad crossing signing and of the crossing itself. Another strategy is to add illumination to passive crossings. In one before-and-after study of 52 highway-rail grade crossings (Russell and Konz, 1980), adding illumination to passive crossings resulted in an 85-percent reduction in the mean number of crashes per week (where the vehicle ran into a train already at the crossing). The importance of illumination at passive highway-rail grade crossings was also highlighted by Fambro (1999).

Mather (1991) evaluated the 7-year crash history of 35 passive highway-rail grade crossings in Oregon that were illuminated as a low-cost alternative for improving crossing safety at night. The eligibility criteria for the installation of illumination were: (1) the crossing must have regular nighttime train movements (4 p.m. to 7 a.m.), and (2) the crossing is too low on the statewide crossing priority list (low train or vehicle traffic volumes) to qualify for automatic warning devices. Crash data indicate that before illumination, 18 train/vehicle crashes occurred at 13 crossings during the hours of darkness. After illumination, only three train/vehicle crashes occurred at two crossings during the hours of darkness.

The cost of installation of the 34 crossings averaged $1,931 per crossing, and ranged from $386 to $9,384 (where a 1.6-km-/ [1-mi-] long ditch was dug to provide electrical power to the site). The goal was $2,000 per crossing (a substantial reduction of the costs associated with installing active warning devices, estimated to be in excess of $100,000 per crossing, according to Bridwell et al., 1993). Monthly maintenance costs averaged $15 per luminaire per pole.

The installation specifications for illumination at the crossings reported on by Mather (1991) are as follows:

- At least one luminaire shall be mounted on each side of the track at the crossing. Luminaires should be located so that protective devices at the crossing will be directly illuminated.

- Luminaires shall be oriented *toward the railroad track* to provide at least 10.76 lux [1 footcandle (fc)] of illumination on the vertical plane 1.5 m (5 ft) from the centerline of the track. Maximum permissible level of illumination and exact orientation of the luminaire will be determined on a case-by-case basis. Factors at the site, including the ambient level of nighttime illumination, need to be considered. The maximum level of illumination is related to the level of lighting on the roadway approaches. The level of illumination should be sufficient to alert drivers to the crossing ahead and to any railroad equipment occupying the crossing, but should not be so bright as to create a blinding effect for motorists in the area immediately beyond the crossing. Cutoffs will normally be used on luminaires to minimize this blinding effect.

- Luminaires should illuminate an area along the track that is 50-percent wider than the traveled width of the road. The illumination should cover a distance equal to the normal height of rail equipment (at least 4.6 m [15 ft] above the top of the rail).

- Poles holding luminaires should be located so that they can be maintained from the roadway right-of-way.

Mather (1991) states that it was difficult to convince highway authorities and electrical companies that the luminaires should be aligned toward the railroad tracks instead of the roadway. Eventually, through meetings and demonstrations, all parties agreed that the luminaires were more effective if they were aligned toward the track. Light readings taken at all tracks showed that a higher percentage of the installations complied with the 10.76-lux (1-fc) standard for illumination where the luminaires faced the railroad tracks. Mather provided the following specifications for the installations. For single-track crossings, poles were located 7.6 m (25 ft) from both the road and the centerline of the railroad track. The 200-W, high-pressure sodium luminaires were placed at least 9.1 m (30 ft) above the top of the rail on 1.8- to 4.9-m-(6- to 16-ft-) long arms. If a railroad signal system was involved, full cutoff luminaires were used. For multiple-track crossings, 400-W, high-pressure sodium luminaires were placed at least 12 m (40 ft) above the top of the rail. If a considerable distance separated the tracks, it was desirable to install a luminaire between the tracks. Semi-cutoff luminaires were used because they spread the light over a larger area of the crossing. This treatment was needed at crossings with three or more tracks, and those with severe angles of intersection.

The largest body of work has been directed toward improving performance under daytime conditions, seeking to improve conspicuity for advance warning devices as well as devices at the crossing, but principally focusing upon motorists' understanding of signs at highway-rail grade crossings. The use of novel sign designs, combinations, materials, and placements has been evaluated, including expanded applications of conventional devices—particularly stop signs.

Research has indicated that driver comprehension of highway-rail grade crossing signs is poor. Even when the signs are conspicuous, they don't provide any information about what drivers should do when approaching or crossing the tracks. The *MUTCD* (1988) specifies that a railroad-crossing sign (Crossbuck, R15-1) is a regulatory sign, and both the 1988 and 2000 editions indicate that at a minimum, one sign shall be used on each roadway approach to every grade crossing, alone or in combination with other traffic control devices. Although the crossbuck is, in effect, a yield sign, and motorists have the obligation to interpret it as such, drivers do not have a clear understanding of what their responsibilities are when encountering the crossbuck sign (Fambro et al., 1997). Lerner, Ratté, and Walker (1990) indicate that across studies, the majority of subjects (54 to 84 percent) believe that the appropriate behavior at a passive crossing is to stop; however, observational studies indicate that drivers routinely disregard this "rule."

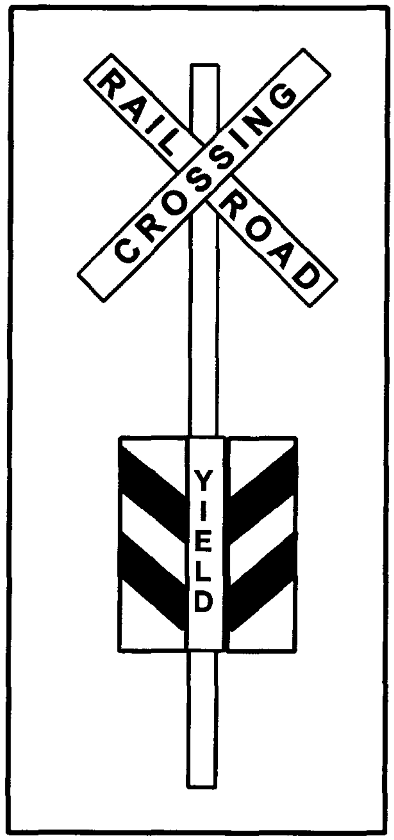

Figure 22. Enhanced crossbuck sign, referred to as the "Buckeye Crossbuck" or "Conrail Shield."

One area of research has attempted to enhance the conspicuity and comprehensibility of the crossbuck sign. An enhanced crossbuck sign is being evaluated in Ohio. The Buckeye crossbuck (also known as the Conrail Shield) is a standard crossbuck sign with a supplemental reflectorized aluminum shield with a red vertical YIELD legend mounted at headlight level. The 950-mm-(38-in-) high shield contains a 225-mm-(9-in-) wide center section (YIELD) with two 300-mm (12-in) side panels that are bent away from the center panel at 45-degree angles (see figure 22). These panels contain alternating stripes of red and white highly retroreflective sheeting, with narrow mirrored strips between the stripes. This device, which costs approximately $300 per crossing, showed significant long-term increases in deceleration rates in Kansas, according to respondents to Fambro's 1999 survey of nine States in the United States. Fambro (1999) recommends the use of this device at passive grade c`rossings in rural areas as an interim device until the research study is complete, based on preliminary findings that conspicuity distance is increased for this sign. Further evaluation data, comparing driver behavior and crashes to an enhanced black-and-white crossbuck, will be provided by the Ohio study, where each treatment is applied at half of the passive crossings in the State.

Bridwell, Alicandri, Fischer, and Kloeppel (1993) conducted a laboratory study that employed 42 young/middle-aged subjects (ages 25 to 45) and 42 older subjects (ages 65 to 85) who viewed slides of 7 railroad crossing signs. The signs included the standard crossbuck sign (R15-1); the standard crossbuck sign with a red-and-white striped (barber) pole; the standard crossbuck sign with a standard yield sign mounted below; the standard crossbuck with the Conrail Shield mounted below; the Canadian Crossbuck (crossbuck with white panel, red border, and no text); the Canadian Crossbuck with the Conrail Shield mounted below; and a standard yield sign with a black-and-white regulatory sign below reading TO TRAINS. Sign recognition distance, conspicuity distance (except for the YIELD TO TRAINS sign), and driver comprehension data were collected. Although there were no significant differences in recognition distance between the signs, the authors suggest that this may be more a result of the testing conditions (laboratory) than of the signs themselves. They recommend that a field study be conducted to measure actual recognition distances. In terms of conspicuity distance, the worst performing signs were the two that contained only the crossbuck (standard R15-1 and the Canadian Crossbuck). The standard crossbuck, when supplemented with the yield sign, or the Conrail Shield, or a barber-striped pole, was noticed significantly more often.

For the comprehension portion of the Bridwell et al. (1993) study, drivers were asked what they thought the sign meant and also what they should do if they encountered such a sign on the roadway. Drivers who didn't know the meaning of a sign (that there is a railroad crossing present) were shown the advanced railroad crossing (W10-1) warning sign to help put the crossbuck sign in context. Several drivers required this "in context" information to correctly identify the meaning of the Canadian Crossbuck, the Canadian Crossbuck with the Conrail Shield, and the YIELD TO TRAINS sign. Data were therefore presented for percent correct for sign meaning before and after being shown the advance railroad crossing sign. Responses were always correct (100 percent comprehension) before the advance sign for three of the signs: (1) the standard crossbuck, (2) the standard crossbuck sign with the barber pole, and (3) the standard crossbuck with the yield sign. For the other four signs, the "before" data for percent correct responses were as follows: (1) standard crossbuck with Conrail Shield (83.3 percent); (2) Canadian Crossbuck (91.7 percent); (3) Canadian Crossbuck with Conrail Shield (58.3 percent); and (4) YIELD TO TRAINS sign (66.7 percent). After presentation of the advanced railroad crossing sign, comprehension improved for three signs: all drivers understood the meaning of the Canadian Crossbuck and the YIELD TO TRAINS sign (100 percent correct comprehension), and comprehension improved from 58.3 percent correct to 83.3 percent correct for the Canadian Crossbuck with the Conrail Shield. In terms of knowing what to do when encountering such a sign, the two best signs (83.3 percent correct for the standard crossbuck with the yield sign and the YIELD TO TRAINS sign) significantly outperformed the two worst signs (the standard crossbuck [41.6 percent correct] and the Canadian Crossbuck [33.3 percent correct]). The other three signs were understood correctly half of the time. After being shown the advance railroad crossing sign, the only sign that showed significant improvement in driver understanding of the correct action was the YIELD TO TRAINS sign, which increased to 100 percent identification of the correct action.

Bridwell et al. (1993) conclude that a change in the current standard crossbuck sign for passive crossings appears to be necessary based on study findings that it is neither well understood nor well noticed. They recommend further testing in the field for the standard crossbuck sign supplemented with the standard yield sign, the standard crossbuck sign supplemented with the Conrail Shield (which has the words YIELD), and the standard yield sign with the supplemental plaque that reads TO TRAINS. The Canadian Crossbuck should be omitted from further testing, as it performed worse than the standard crossbuck.

Fambro (1999) conducted five focus groups containing 10 participants each, who were selected to provide a range of ages. Focus groups were conducted in College Station, TX; Corsicana, TX; Arlington, TX; De Kalb, IL; and St. Charles, IL. During the sessions, participants were asked to provide their reactions to the following eight traffic control devices described as proposed enhancements to passive warning devices at highway-rail grade crossings: (1) Buckeye Crossbuck; (2) LOOK FOR TRAINS sign; (3) YIELD TO TRAINS sign; (4) a vehicle-activated strobe light; (5) illumination; (6) rumble strips; (7) additional retroreflective material and/or devices on crossbuck and support posts; and (8) roadway traffic signals. None of the passive signing systems was recommended by focus group participants as promising, although the Buckeye Crossbuck was rated as an excellent alternative in one of the five focus groups.

In the Fambro et al. (1997) survey of 1,010 drivers conducted to determine driver understanding of traffic control devices at highway-rail crossings, 82 percent of the respondents correctly identified the meaning of the YIELD TO TRAINS experimental sign (i.e., "Yield the right-of-way if a train is approaching a crossing"). In the focus group sessions conducted by Fambro (1999), the YIELD TO TRAINS signs were not rated as promising alternative traffic control devices. However, he recommends their use at passive grade crossings in rural areas to alert and warn drivers that they are approaching a critical safety decision point. The sign consists of a standard yield sign with a supplemental panel with the phrase, TO TRAINS. The sign combination can be placed on the standard crossbuck sign or used on a separate pole at the crossing. Fambro's recommendation is based on a before-and-after study conducted in Texas, which found a significant decrease in approach speed at two of six sites; significant increases in looking behavior at three of eight sites; and no significant decrease in looking behavior at any of the eight sites evaluated (Fambro, Beitler, and Hubbard, 1994).

The above recommendations with combinations involving the standard R1-2 sign focuses attention on the use of traffic control devices whose meanings are familiar to drivers. The *MUTCD* (2000) indicates that stop or yield signs may be used at highway-rail intersections at the discretion of the responsible State or local jurisdiction for crossings that have two or more trains per day and are without automatic traffic control devices. It further states that for other crossings with passive protection, stop or yield signs may be used after a need is established by a traffic engineering study and may be placed on the crossbuck post. Engineering studies should take into account such factors as highway and train traffic characteristics (volume and speed), crash history, the need for active control devices, and sight distance to the approaching train.

The use of stop signs at passive crossings has been a controversial issue for more than 40 years (Russell and Burnham, 1999). One camp (including Russell and Burnham) maintains that the indiscriminate use of stop signs at all passive crossings would serve to breed driver disrespect for stop signs, as well as for highway-rail grade crossings. Russell and Burnham (1999) cite the research of Bezkorovainy-Holsinger (1966) and Burnham (1994) who observed that approximately 84 percent of drivers at stop-controlled highway-rail grade crossings violated the law and either did not stop at all, or performed a rolling stop.

This view is contrasted against that of the National Transportation Safety Board (NTSB), who, in 1998, recommended that States install stop signs at all passive grade crossings unless a traffic engineering analysis determines that installation of a stop sign would reduce the level of safety at a crossing. Farr and Hitz (1985) found that stop signs reduced crossing crashes by approximately 35 percent, based on analysis of data included in the U.S. Department of Transportation (DOT)/Association of American Railroads (AAR) *National Rail-Highway Crossing Inventory* and the Federal Railroad Administration (FRA) *Railroad Accident/Incident Reporting System* for the years 1975 through 1980. Eck and Shanmugam (1987) used the *National Rail-Highway Crossing Inventory* and FRA crash files to compare low-volume road grade crossing characteristics with high-volume road grade crossing characteristics. They found that exposure-based crash rates at low-volume road grade crossings were much higher than at higher volume road grade crossings, and that magnitudes of crash reductions following upgrades from no signs or crossbucks only to stop signs were higher for the low-volume crossings.

Earlier work in this area has also influenced current thinking, particularly the Sanders, McGee, and Yoo (1978) study, which determined the advantages and disadvantages of the selective use of highway stop signs as safety improvements at highway-rail grade crossings and developed guidelines for their use. They performed crash analyses to compare crash rates for crossings with crossbucks only to crash rates for crossings with crossbucks and standard highway stop signs. Results indicated that crash rates for stop-sign crossings were lower than the rates for Crossbuck-only crossings, given higher vehicle/train exposure. Sanders et al. (1978) also performed field studies to compare driver behavior for crossbuck-only crossings to driver behavior for similar crossings where a standard highway stop sign was installed in addition to the Crossbuck. Driver behavior included looking behavior, speed profiles, and observance of stop signs. Looking behavior was obtained by Sanders et al. (1978) for 1,413 drivers at 8 crossbuck-only sites in 4 States, and for 3,073 drivers at 18 sites with both a stop sign and a crossbuck sign in 5 States. They found that 83 percent of the drivers looked for trains at the locations with stop signs, but only 42 percent looked for trains at the crossbuck-only sites. Speed profile measures indicated that drivers approaching a stop-sign crossing began their deceleration closer to the crossing than did drivers approaching a Crossbuck crossing. Also, vehicle speeds just prior to the crossing were considerably lower at the STOP-sign crossing. Sanders et al. (1978) observed that while stop-sign observance (compliance) at stop-controlled highway-rail crossings was less than at highway intersections (60 percent versus 80 percent), there was no transfer of adverse stopping behavior between the rail-grade crossing and a nearby stop-controlled highway intersection.

Applying the results of Sanders et al. (1978) to the goal of accommodating age-related driver difficulties is hampered by lack of knowledge about the age of the drivers sampled in their research. In addition, the driver behaviors adopted as outcome variables were affected by factors other than signing practices and site characteristics. For example, looking behavior was affected by the level of enforcement at stop-sign-controlled crossings. Their principal conclusion still is worth noting: Stop signs should be applied selectively only at hazardous passive grade crossings (restricted sight triangle; ADT<2000; 3+ train crossings per day) and should not be used indiscriminately at all passive grade crossings.

More recently, an investigation including seniors was performed by Fambro, Shull, Noyce, and Rahman (1997). This study was predicated upon the belief that looking behavior in *both* directions and a significant reduction in speed are the key safety goals at passive crossings, because no warning devices are activated when a train is approaching the crossing. Fambro et al. (1997) conducted a field study using 10 younger drivers (ages 18 to 25), 10 middle-aged drivers (ages 30 to 45), and 10 older drivers (ages 55 and older), to document looking behavior and deceleration at rail grade crossings. The passive site with the greatest percentage of drivers who looked in both directions before crossing (97 percent) was a site controlled by a stop sign. All of the young and middle-aged drivers looked both ways at this site, and 9 of the 10 older drivers looked both ways. Three of the 30 drivers did not stop at the stop sign (although they slowed considerably), and 10 drivers performed a rolling stop. At the other two passive sites (marked with crossbuck signs only, and no advance railroad crossing sign), 70 percent of the drivers *did not* look in both directions at one site (70 percent of the younger, 80 percent of the middle-aged, and 60 percent of the older drivers), and 17 percent *did not* look in both directions at the other site (20 percent of the younger, 10 percent of the middle-aged, and 20 percent of the older drivers).

It may be noted that stopping before reaching the tracks provides the driver, who may have difficulty dividing attention (e.g., the older driver), with the ability to focus on the single task of looking for a train. For example, Lerner, Ratté, and Walker (1990) note that in the Knoblauch et al. (1982) study, relatively few crashes occurred at a roadway-rail grade crossing because a driver who was stopped (at a passive crossing or at an active crossing with flashing lights) misjudged a gap and proceeded to cross. The large majority of decision errors were made by drivers in moving cars. Expecting to stop, and seeing a stop sign at a passive crossing, would also remove the potential for vehicle-to-vehicle (rear-end) crashes. It is relative that 15 to 25 percent of drivers expect all crossings to be actively controlled (Fambro, 1999; Lerner et al., 1990). This erroneous perception results in drivers assuming that it is safe to cross (without slowing or looking both ways) when there are no lights flashing, even when there are no lights present at the crossing. A stop sign at a passive crossing will eliminate this expectancy and provide positive and unambiguous guidance, removing the need to look for the *absence* of active warnings for clues about what behavior is appropriate. Also, as Lerner et al. (1990) note, response time is faster to detect the presence of a traffic control device as opposed to its absence.

Together, the findings and conclusions in the paragraphs above should provide guidance for the selective use of the R1-1 sign at highway-rail grade crossings by alerting the engineering judgment called out in the *MUTCD* for application of these devices.

As described in chapter I (Design Element L: Stop- and Yield-Controlled Intersection Signing), highway signs with fluorescent sheeting have been found to be more conspicuous and can be detected at a farther distance than signs with standard sheeting of the same color. Of particular interest is a study by Burns and Pavelka (1995), that found that signs with fluorescent red sheeting had greater detection and color recognition distances at dusk than signs made with standard red sheeting. The results of this study suggest that the use of fluorescent red sheeting on yield signs at highway-rail grade crossings (and on stop signs, where they are deemed appropriate), would serve to increase their conspicuity, both during the day and under low-luminance conditions, and would be of particular benefit to older drivers who suffer from a decrease in contrast sensitivity.

Other research pertaining to signing for highway-rail grade crossings for which data from older drivers have been obtained has addressed comprehension of the railroad advance warning sign and the parallel railroad advance warning sign. Picha, Hawkins, and Womack (1995) conducted a survey of 747 drivers ranging in age from 16 to 65 years and older who were renewing their driver's licenses in seven Texas cities. Of the 747 participants, 54 were ages 55 to 64 (7.3 percent of the sample) and 31 were age 65 or older (4.2 percent of the sample). A multiple-choice question was included regarding the meaning of the W10-1 (Railroad Advance Warning) sign and the W10-3 (Parallel Railroad Advance Warning) sign. No advantages for alternative designs for the standard W10-1 were demonstrated in this research; however, an alternative to the current W10-3 was recommended.

The standard Parallel Railroad Advance Warning sign (W10-3) and three alternative designs were shown to the same driver sample (see figure 23). Alternative 1 was a yellow diamond sign that consisted of the same elements present in the standard design, except that the roadway outline was drawn (as opposed to thick solid lines), and a bent right arrow was drawn within the roadway lines to indicate that a right turn would lead to railroad tracks. Alternative 2 was the standard W10-1 sign (Railroad Advance Warning) with a supplemental plaque containing an arrow that pointed to the right. Alternative 3 was the same as Alternative 2, except the supplemental panel contained a bent right-pointing arrow. The correct response, "you will cross a railroad track if you turn right at the intersection," was provided by 84.1 percent of the participants who saw the standard sign; 88.1 percent of the respondents who saw Alternative 1; 90.5 percent of the respondents who saw Alternative 2; and 87.2 percent of the respondents who saw Alternative 3. A higher percentage of respondents indicated that they did not know what the standard sign meant (10.2 percent) than the alternative designs (6.2 percent for Alternative 1; 3.2 percent for Alternative 2; and 1.6 percent for Alternative 3). Thus, the standard W10-3 sign had the lowest correct response rate and the highest "not sure" rate, although these differences did not reach statistical significance. While suggestive, further work is deemed necessary to justify a recommendation in this Handbook.

The next category of countermeasures reviewed targeting speed reductions by drivers approaching highway-rail grade crossings is the application of rumble strips. Fambro's (1999) review indicates that in Kentucky, rumble strips were effective in reducing collisions and near misses, with no indication of motorists avoiding the rumble strips; however, in Georgia, about

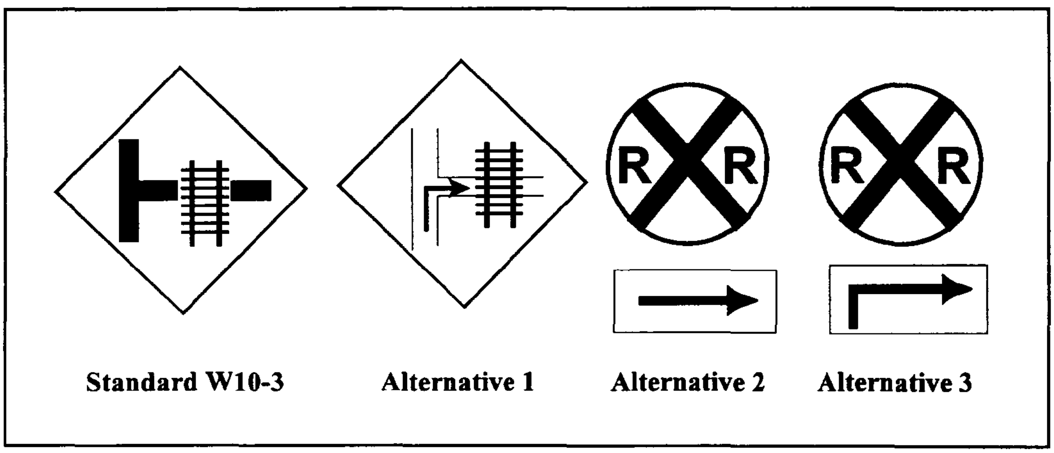

Figure 23. Standard W10-3 sign and alternative sign designs evaluated by Picha, Hawkins, and Womak (1995).

12 drivers per day "drove around" the strips (Skinner, 1971; Parsonson and Rinalducci, 1982). Parsonson and Rinalducci (1982) recommend that at grade crossings, rumble strips should be applied only at non-residential locations where unfamiliar drivers are the prevalent group. Fambro states that the potential benefits of rumble strips at roadway-rail grade crossings include: decreased approach speeds; increased awareness of warning signs; and increased awareness of a potentially hazardous intersection. He recommends their use for passive crossings on rural, low-volume roadways and provides the following guidelines:

- Three to four rumble strip pads of 5 to 20 rumble strips should be used, with the first pad placed 2 to 3 s before the advance warning sign to direct the driver's attention to the sign.
- Subsequent pads should be placed with decreasing spacing and numbers to create the sensation of acceleration.
- The last pad should be placed at least 76 m (250 ft) before the crossing to avoid creating a pavement condition that might interfere with braking.

Another view is provided by Lerner, Ratté, and Walker (1990), who caution that distorting the driver's perception of approach speed with progressively decreasing spacings of rumble strips in an effort to encourage their slowing on the approach to a crossing, could actually encourage a driver to speed up and try to beat a train. While this behavior has not typically been associated with seniors, either in the research literature or anecdotally, it raises a more general concern that any present recommendation regarding the use of rumble strips would be premature.

Finally, a novel treatment to attract motorists' attention to highway-rail grade crossings is the use of a vehicle-activated strobe light on a sign assembly. The vehicle-activated strobe light causes short bursts of flashing light when a vehicle passes over detectors placed in the roadway. In research conducted by the Texas Transportation Institute, the strobe is mounted on top of a standard railroad advance warning sign, and the vehicle detector is placed approximately 8 s before the advance warning sign (Fambro, Schull, Noyce, and Rahman, (1997). The strobe flashes three to five times to direct the driver's attention toward the advance warning sign, and it should stop flashing 2 to 3 seconds before the driver reaches the sign to allow time to read the sign.

Fambro et al. (1997) evaluated this treatment in a controlled field study using 7 younger drivers (age 25 or younger), 12 middle-aged drivers (ages 25 to 54), and 7 older drivers (age 55 or older). Head movement toward the sign and braking reactions were recorded by in-vehicle observers for the advance railroad crossing sign alone, the advance railroad crossing sign supplemented with a strobe light, and an advance railroad warning sign supplemented with a standard flashing beacon. The sign sheeting was engineering grade, making it retroreflective. There were no differences in head movement (indicating no differences in attention-getting value) as a function of the sign system; however, 54 percent of the drivers exhibited braking in response to signs enhanced with either the strobe light or the flashers, compared to 31 percent who exhibited braking for the standard sign without enhancement. Drivers also responded to a questionnaire and participated in a focus group discussion. Three drivers (11 percent) thought that the strobe-enhanced sign indicated that a train was ahead and nine drivers (35 percent) thought that the flashing beacon indicated the presence of a train. The authors mention a concern with this interpretation that drivers at passive crossings who do not see trains approaching after encountering strobe- or flasher-enhanced warning signs may begin to disrespect active devices at crossings that do indicate the presence of a train. Seven drivers (27 percent) indicated that the strobe light was confusing and 11 drivers (42 percent) thought that the flasher-enhanced sign was confusing. With special attention being paid to the problems of older drivers, including the difficulties in decision-making and delays in response time when confronted with unusual or unexpected situations, such results were not sufficiently encouraging to warrant a recommendation in this Handbook.

GLOSSARY

AAAFTS. American Automobile Association Foundation for Traffic Safety.

AADT. Annual average daily traffic.

AASHTO. American Association of State Highway and Transportation Officials.

ADT. Average daily traffic.

Ambient conditions. The visual background or surrounding atmospheric and visibility conditions.

Angular motion sensitivity. The ability of an observer to detect changes in the apparent distance and direction of movement of an object as a function of the change in the angular size of the visual stimulus on the observer's retina.

Angular velocity threshold. The rate of change in angular size of a visual stimulus that is necessary for an observer to discern that an object's motion has increased or decreased.

Annual average daily traffic (AADT). The total volume passing a point or segment of a highway facility in both directions for 1 year divided by the number of days in the year.

Apron. The mountable portion of the central island of a roundabout that is adjacent to the circulatory roadway. An apron is generally required on smaller roundabouts to accommodate the wheel tracking of large vehicles.

ASTM. American Society for Testing and Materials.

ATSSA. American Traffic Safety Services Association.

Attraction signing. Information/supplemental signs featuring logos or verbal messages pointing out places to visit or food, gas, and rest stop locations.

Barnes Dance timing. Type of exclusive signal-timing phase where pedestrians may also cross diagonally in addition to crossing either street. Also referred to as scramble timing.

Brake reaction time. The interval between the instant that the driver recognizes the presence of an object or hazard on the roadway ahead and the instant that the driver actually applies the brakes.

Brightness. A term that refers to human perception of luminance. Whereas luminance is a photometrically measured quantity, brightness describes how intense a light source or lighted surface appears to the human eye.

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Buttonhook ramp. J-shaped ramp that connects to a parallel or diagonal street or frontage road, which is often well removed from the interchange structure and other ramps.
Candela (cd). A measure of luminous intensity.

Central island. The raised area in the center of a roundabout around which traffic circulates.

Changeable message sign (CMS). Sometimes called portable changeable or variable message sign. This traffic control device has the flexibility to display a variety of messages to fit the needs of the traffic and highway situation.

Channelization. The separation or regulation of conflicting traffic movement into definite paths of travel by the use of pavement markings, raised islands, or other suitable means, to facilitate the safe and orderly movement of both vehicles and pedestrians.

Chevron signs. A chevron symbol (sideways "V") in black, against standard yellow background, on a vertical rectangle. Used as an alternate or supplement to standard delineators and to large arrow signs.

CIE. Commission Internationale de l'Éclairage (International Commission on Street/Highway Lighting).

Circulatory roadway. The curved path used by vehicles to travel in a counterclockwise fashion around the central island of a roundabout.

Circulatory roadway width. The width between the outer edge of the circulatory roadway and the central island, not including the width of any apron.

Cloverleaf interchange. A form of interchange that provides indirect right-turn movements in all four quadrants by means of loops. Generally used where the turning and weaving volumes are relatively low. This type of interchange eliminates all crossing conflicts found in a diamond interchange, but requires more area. The cloverleaf type of interchange can have one or two points of entry and exit on each through roadway.

Coefficient of luminous intensity (R_I). The ratio of the luminous intensity (I) of a retroreflectometer in the direction of observation to the illuminance $E_\perp$ at the retroreflectometer on a plane perpendicular to the direction of the incident light, expressed in candelas per lux.

Coefficient of retroreflected luminance (R_L). A measure of retroreflection most often used to describe the retroreflectivity of pavement markings. Coefficient of retroreflected luminance is defined as the coefficient of luminous intensity per unit area.

Coefficient of retroreflection (R_A). A measure of retroreflection used more often to refer to the retroreflectivity of highway signs. Coefficient of retroreflection is defined as the ratio of the coefficient of luminous intensity (R_I) of a plane retroreflecting surface to its area (A), expressed in candelas per lux per square meter.

Complete interchange lighting (CIL). Includes lighting in the interchange area on both the acceleration and deceleration areas, plus the ramps through the terminus.

Composite photometry. Light measurement applied to a high-mast lighting system that employs a counterbeam arrangement to take advantage of the efficiency with which pavement luminance can be increased with light directed upstream, while enhancing positive contrast through additivity of vehicle headlighting with the light directed downstream.

Concrete safety-shaped barrier (CSSB). A tapered concrete barrier used as a highway divider in narrow medians to prevent vehicle crossovers into oncoming traffic. It is referred to as a *Jersey barrier* in some jurisdictions, as its first application was on the New Jersey Turnpike. CSSBs can be either permanent barriers or temporary portable barriers used in work-zone applications.

Conspicuity. A measure of the likelihood that a driver will notice a certain target at a given distance against a certain background.

Contrast. *See* **Luminance contrast.**

Contrast sensitivity. Ability to perceive a lightness or brightness difference between two areas. Frequently measured for a range of target patterns differing in value along some dimension, such as pattern element size, and portrayed graphically in a contrast sensitivity function in which the reciprocal of contrast threshold is plotted against pattern spatial frequency or against visual angle subtended at the eye by pattern elements (such as bars).

Critical gap. The gap (distance to nearest vehicle) in oncoming or cross-traffic that a driver will accept to initiate a turning or crossing maneuver 50 percent of the time it is presented, typically measured in seconds.

Crossbuck. White X-shaped retroreflectorized highway-rail grade crossing sign with the words RAILROAD CROSSING in black lettering, located alongside the highway at the railroad tracks. At multiple-track crossings, a sign indicating the number of tracks will be on the post of the crossbuck.

Cutoff. A luminaire light distribution is designed as a cutoff when the candlepower per 1000 lamp lumens does not numerically exceed 25 (2.5 percent) at an angle of 90 degrees above nadir (horizontal) and 100 (10 percent) at a vertical angle of 80 degrees above nadir. This applies to any lateral angle around the luminaire.

Dark adaptation. Adjustment of the eye to low levels of illumination, which results in increased sensitivity to light.

Decision sight distance (DSD). The distance required for a driver to detect an unexpected or otherwise difficult-to-perceive information source or hazard in a roadway environment that may be visually cluttered, recognize the hazard or its threat potential, select an appropriate speed and path, and initiate and complete the required safety maneuver safely and efficiently.

Deflection. The change in the trajectory of a vehicle imposed by the geometric features of the roadway.

Depth perception. The ability to distinguish the relative distance of objects in visual space, used to interpret their motion over multiple observations.

Diamond interchange. The simplest and perhaps most common type of interchange. This type of interchange contains a one-way, diagonal-type ramp in one or more of the quadrants. The diamond interchange provides for all movements to and from the intersecting road.

Diverge steering (DS) zone. Used with interchange/ramp exit models, it is the distance upstream from the exit gore at which a driver begins to diverge from the freeway.

Divided attention. The ability of a driver to allocate attention among tasks or stimuli in the roadway environment, where more than one task or stimulus is perceived to be important to safe performance at a given time.

Divided highway. Roadway that is separated by a median.

Downstream. The direction toward which traffic is flowing.

Dynamic visual acuity. Acuteness or clarity of vision for an object that has angular movement relative to the observer. Acuity depends on sharpness of retinal focus, sensitivity of nervous elements, oculomotor coordination, interpretative faculty of the brain, and contextual variables.

Edgeline visibility. The detection/recognition of painted pavement surface delineation along roadway edges.

Entry width. The width of the entry to a roundabout, where it meets the inscribed circle, measured perpendicularly from the right edge of the entry to the intersection point of the left edgeline and the inscribed circle.

Exit gore area. The area located immediately between the left edge of a ramp pavement and the right edge of the mainline roadway pavement at a merge or diverge area.

FARS. Fatal Analysis Reporting System

FHWA. Federal Highway Administration.

Flared approach. The widening of an approach to a roundabout, resulting in multiple lanes at entry to provide additional capacity at the yield line and storage.

Footcandle. The English system's unit of illuminance, equivalent to the illumination produced by a source of one candle at a distance of 0.305 m (1 ft) and equal to 1 lumen incident per 0.09 m^2 (1 ft^2). One footcandle equals 10.76 lux.

Footlambert. A unit of luminance equivalent to 1 lumen per 0.09 m^2 (1 ft^2).

Full-diamond interchange. Interchange with a one-way, diagonal-type ramp in each quadrant.

Gap acceptance. The decision by a driver that there is sufficient time and/or distance ahead of an approaching vehicle to allow safe performance of a desired crossing or merging maneuver.

Gap judgments. The judgment of a driver of the time and/or distance ahead of an approaching vehicle traveling in a lane that the driver wishes to turn across or merge into.

Gap search and acceptance (GSA) zone. Used with interchange/ramp entry models, it is the zone in which the driver searches, evaluates, and accepts or rejects the available lags or gaps in the traffic stream for execution of a merging maneuver.

Guardrail (guiderail). Protective barrier along a roadway to prevent vehicles from leaving the roadway.

Half-diamond interchange. An interchange with a one-way, diagonal-type ramp in two adjacent quadrants. This type of interchange is appropriate to situations in which traffic demand is predominantly in one direction.

High-mast lighting. Illumination of a large area by means of a cluster of 3 to 12 luminaires that are designed to be mounted in a fixed orientation at the top of a high mast (generally 18 to 46 m [80 to 150 ft] or higher).

High-spatial-frequency stimulus. A visual target characterized by fine detail.

Highway-rail grade crossing. The general area where a highway and a railroad's right-of-way cross at the same level, including the railroad tracks, highway, and traffic control devices for highway traffic traversing the area.

Horizontal alignment. The linear (tangent) character or specific degree of curvature describing the geometry of a defined section of highway in plan view.

IIHS. Insurance Institute for Highway Safety.

Illuminance. The density of luminous flux (rate of emission of luminous energy flow of a light source in all directions) incident on a surface; the quotient of the flux divided by the area of the surface, when the surface is uniformly illuminated.

Illumination. The amount of light falling onto a surface.

Initial acceleration (IA) zone. Used with interchange/ramp entry models, it is the zone in which the driver accelerates to reduce the speed differential between the ramp vehicle and the freeway vehicles to an acceptable level for completing the merge process.

Inscribed circle diameter. The basic parameter used to define the size of a roundabout, measured between the outer edges of the circulatory roadway. It is the diameter of the largest circle that can be inscribed within the outline of the intersection.

In-service brightness level (ISBL). The brightness level of a delineation treatment at an intermediate point in its anticipated service life; this value varies by type of delineator, type of wear (traffic level), and environmental conditions.

Interchange (grade separation). A system of interconnecting roadways that provides for the movement of traffic between two or more highways on different levels.

Intersecting angle (skew). The angle formed by the intersection of two roadways (other than a 90-degree angle).

Intersection (at grade). The general area where two or more highways join or cross without grade separation, including the roadway and roadside facilities for traffic movements within it.

Intersection sight distance (ISD). The unobstructed view of an entire (at-grade) intersection and sufficient lengths of the intersecting highway to permit control of the vehicle to avoid collisions during through and turning movements.

ISTEA. Intermodal Surface Transportation Efficiency Act.

ITE. Institute of Transportation Engineers.

Joint flexibility. An aspect of the physical condition of the driver that can be assessed to determine whether the driver has sufficient strength to turn the steering wheel, apply the brakes, and generally control the vehicle.

Lane-control signals. Special overhead signals that permit or prohibit the use of specific lanes of a street or highway or that indicate the impending prohibition of their use.

Leading pedestrian interval (LPI). Also known as "pedestrian head start" and "delayed vehicle green," an LPI allows pedestrians to begin crossing an intersection a few seconds before the vehicular green interval begins. This allows pedestrians to establish their presence in the crosswalk before the turning vehicles, thereby enhancing the pedestrian right-of-way.

Legibility index (LI). Used to describe the relative legibility of different letter styles, it is calculated from the distance at which a character, word, or message is legible divided by the size of the letters on the sign.

Limited sight distance. A restricted preview of the traveled way downstream due to a crest vertical curve or horizontal curvature of the roadway, or to blockage or obstruction by a natural or man-made roadway feature or by (an)other vehicle(s).

Luminaire. A complete lighting unit consisting of a lamp or lamps, together with the parts designed to distribute the light, to position and protect the lamps, and to connect the lamps to the power supply.

Luminance. The luminous intensity or brightness of any surface in a given direction, per unit of projected area of the surface as viewed from that direction, independent of viewing distance. The SI unit (metric) is candela per square meter.

Luminance contrast. The difference between the luminance of a target area and a surrounding background area, divided by the background luminance alone (e.g., lane marking minus lane pavement surface, divided by pavement surface.)

Lux. The metric unit of illuminance. One lux is equal to the illuminance corresponding to a luminous flux density of 1 lumen per square meter.

Measures of effectiveness (MOEs). Descriptions of driver or traffic behavior that quantify the level of safety or the quality of service provided by a facility or treatment to drivers, passengers, or pedestrians; examples include vehicle speed, trajectory, delay, and similar measures, especially crashes, plus indices of performance such as reaction time. In research studies, the MOEs are the dependent measures (e.g., the effects/behaviors resulting from introduction of a treatment or countermeasure).

Median barriers. A longitudinal system of physical barriers used to prevent an errant vehicle from crossing the portion of a divided highway separating traffic moving in opposite directions.

Merge steering control (MSC) zone. Used with interchange/ramp entry models, it is the zone in which the driver enters the freeway and positions the vehicle in the nearest lane on the mainline.

Minimum required visibility distance (MRVD). The distance necessary to permit detection and comprehension, plus driver decision-making, response selection, and completion of a vehicle maneuver, if necessary.

Mountable. Geometric features (e.g., curbs) that can be driven upon by vehicles without damage, but are not intended to be in the normal path of traffic.

MUTCD. Manual on Uniform Traffic Control Devices for Streets and Highways.

NCHRP. National Cooperative Highway Research Program.

Nearside priority. Priority given to drivers entering the circle of a roundabout.

Negative offset. A term used to describe the alignment of opposing left-turn lanes at an intersection; this geometry exists when the left boundary of one left-turn lane, when extended across the intersection, falls to the right of the right boundary of the opposite left-turn lane.

NHTSA. National Highway Traffic Safety Administration.

Non-cutoff. The luminaire light distribution category when there is no candlepower limitation in the zone above maximum candlepower.

No turn on red (NTOR). This message on signs is used to indicate that a right-turn-on-red (or left-turn-on-red for one-way streets) is not permitted at an intersection.

NTSB. National Transportation Safety Board.

Ocular media. The internal structure of the eye, including the aqueous region, through which light entering through the cornea must be transmitted before reaching the photosensitive retina.

Ocular transmittance. The amount of light reaching the retina relative to the amount incident upon the cornea.

Offside priority. Priority given to traffic already in the circle of a roundabout.

Osteoarthritis. A degenerative form of arthritis.

Parclo loop ramp. A (partial cloverleaf) interchange with loops in advance of the minor road with direction of travel on the freeway; and in the same interchange area, an interchange with loops beyond the minor road.

Partial interchange lighting (PIL). Lighting on an interchange that consists of a few luminaires located in the general areas where entrance and exit ramps connect with the through traffic lanes of a freeway (between the entry gore and the end of the acceleration ramp or exit gore and the beginning of the deceleration ramp).

Passive crossing control devices. Non-activated traffic control devices, including signs, pavement markings, and other devices located at or in advance of crossings to indicate the presence of a crossing and the possibility of a train.

Peak intensity. The maximum strength of a traffic signal maintained through a defined viewing angle (measured in candelas).

Pedestrian control device. A special type of device (including pedestrian signal indications and sign panels) intended for the exclusive purpose of controlling pedestrian traffic in crosswalks.

Pedestrian crosswalk. An extension of a sidewalk across an intersection or across a roadway at a midblock location to accommodate pedestrian movement.

Pedestrian refuge. An at-grade opening within a median island that allows pedestrians to safely wait for an acceptable gap in traffic.

Perception-reaction time (PRT). The interval between a driver's detection of a target stimulus or event and the initiation of a vehicle control movement in response to the stimulus or event.

Positive offset. A term used to describe the alignment of opposing left-turn lanes at an intersection; this geometry exists when the left boundary of one left-turn lane, when extended across the intersection, falls to the left of the right boundary of the opposite left-turn lane.

Post-mounted delineators (PMDs). Retroreflective devices located serially at the side of a roadway to indicate alignment. Each delineator consists of a flat reflecting surface, typically a vertical rectangle, mounted on a supporting post.

Raised. Geometric features (e.g., curbs) with a sharp elevation change that are not intended to be driven upon by vehicles at any time.

Raised pavement markers (RPMs). Used as positioning guides and/or as supplements or substitutes for other types of markings, these markers conform to the color of the marking for which they serve as a positioning guide, can be mono- or bi-directional, and are fastened into the pavement with the reflector surface visible above the road surface.

Reaction time (RT). The time from the onset of a stimulus to the beginning of a driver's (or pedestrian's) response to the stimulus by a simple movement of a limb or other body part.

Retroreflective. Capable of returning light to its source.

Rheumatoid arthritis. A usually chronic disease of unknown cause characterized by pain, stiffness, inflammation, swelling, and sometimes destruction of joints. Drivers with this condition sometimes require compensatory equipment for their vehicle. In acute conditions, individuals should not drive because of weakness and extreme tenderness in the joints of the wrists and hands.

Right-turn-on-red (RTOR). Unless otherwise specified by traffic signal control signing, this practice permits a driver to proceed with a right turn on a red signal after stopping at signalized intersections. It provides increased capacity and operational efficiency at a low cost.

Roundabouts. Circular intersections with specific design and traffic control features that include yield control of entering traffic, channelized approaches, and appropriate geometric curvature to ensure that travel speeds on the circulating roadway are typically less than 50 km/h (30 mi/h).

Route Marker Reassurance Assembly. Consists of a cardinal direction marker (i.e., east, west, north, and south) and a route marker.

Saccadic movement. A change in visual fixation from one point to another by means of a quick, abrupt movement of the eye.

Scissors off-ramp. A condition where one-way traffic streams cross by merging and diverging maneuvers onto exit ramps. Drivers tend to go straight ahead onto an off-ramp instead of turning left.

Selective attention. The ability, on an ongoing moment-to-moment basis while driving, to identify and allocate attention to the most relevant information, especially embedded when within a visually complex scene and in the presence of a number of distractors.

Senile miosis. An aging characteristic involving an excessive smallness or contraction of the pupil of the eye.

Short-range delineation. Delineation that is useful to the driver for tracking the roadway at night under poor visibility conditions.

Sight distance. The length of highway visible to the driver.

Sight triangle. In plan view, the area defined by the point of intersection of two roadways, and by the driver's line of sight from the point of approach along one leg of the intersection, to the farthest unobstructed location on another leg of the intersection.

Situational awareness. The selective attention to and perception of environmental elements within a specified space and time envelope, the comprehension of their meaning, and the projection of their status in the near future.

Slip ramp. A diagonal ramp, more properly called a cross-connection, which connects with a parallel frontage road.

Small target visibility (STV). A proposed criterion for roadway lighting. The concept assumes that increased target visibility results in both increased nighttime safety and improved nighttime driver performance, a surrogate for reduced crash risk.

Speed-change lane (SCL). Used in interchange/ramp exit models, it refers to the speed-change maneuver on a deceleration lane's segmented components.

Splitter island. A raised or painted area on an approach to a roundabout used to separate entering from exiting traffic, deflect and slow entering traffic, and provide storage space for pedestrians crossing the road in two stages. It is also referred to as a *median island* or *separator island.*

Steering control (SC) zone. Used with interchange/ramp entry models, it is the zone where positioning of the vehicle along a path from the controlling ramp curvature onto the speed-change lane is accomplished.

Stereopsis. Binocular visual perception of three-dimensional space based on retinal disparity.

Stopping sight distance (SSD). The sight distance required to permit drivers to see an obstacle soon enough to stop for it under a defined set of reasonable worst-case conditions, without the performance of any avoidance maneuver or change in travel path. The calculation of SSD depends upon speed, gradient, road surface and tire conditions, and assumptions about the perception-reaction time of the driver.

Temporary pavement marking treatment. This treatment primarily involves the application of paint or tape striping and has been shown to be important for effective vehicle guidance at highway work sites.

Threshold contrast. The minimum difference in the luminance of a target and the luminance of that target's background at which the target is visible. Also defined as the luminance contrast detectable during some specific fraction of the times it is presented to an observer, usually 50 percent.

T-intersection. An intersection that involves three legs, where one leg is perpendicular to the other two legs. There are several types of this intersection, such as plain, with turning lanes, and channelized.

Traffic control device (TCD). The prime, and often the only, means of communicating with the driving public. These devices (e.g., signs, markings, signals, islands) must be used discriminately, uniformly, and effectively to ensure correct driver interpretation and response.

Transient adaptation factor. A reduction in target contrast caused by the process of transient visual adaptation.

Transient visual adaptation (TVA). The process in which the (driver's) eye fixates upon roadway locations or surrounding environments at different luminance levels, continuously adapting to higher and lower levels; this process temporarily reduces contrast sensitivity.

TRB. Transportation Research Board.

Trumpet interchange. A three-leg interchange where a connecting highway terminates and where only a small amount of traffic moves between the terminating highway and one of the two legs of the freeway. The trumpet is laid out so that this minor traffic moves via a 200-degree loop.

TWLTL. Two-way, left-turn lane.

Two-quadrant cloverleaf interchange. A type of partial cloverleaf where most traffic leaving one highway turns to the same leg of the intersecting highway.

Two-way stop control. Stop signs are present on the approaches of the minor street and drivers on the minor street (or a driver turning left from the major street) must wait for a gap in the major-street traffic to complete a maneuver.

Upstream. The direction from which traffic is flowing.

Useful field of view. Also known as the "functional field of view" or the "attentional window," this area refers to a subset of the total field of view where stimuli can not only be detected, but can be recognized and understood sufficiently to permit a timely driver response. As such, this term represents an aspect of visual information processing, rather than a measure of visual sensitivity.

Variable message sign (VMS). *See* **Changeable message sign**.

Veiling glare. Stray light entering the eye that reduces the contrast of a target upon which the driver has fixated; this may result from the driver's direct view of light sources, such as opposing headlights or roadway luminaires, or from light reflected from surfaces near the target's location.

Vertical curve. The parabolic curve connecting the two approach grades on either side of a hill.

Visual accommodation. The process by which the eye changes focus from one distance to another.

Visual acuity. The ability of an observer to resolve fine pattern detail. Acuity is usually specified in terms of decimal acuity, defined as the reciprocal of the smallest resolvable pattern detail in minutes of arc of visual angle. "Normal" or average acuity is considered to be 1.0 (a resolution of 1-min arc).

Visual adaptation. The process by which the retina becomes accustomed to more or less light than it was exposed to during an immediately preceding period. It results in a change in the sensitivity of the eye to light.

Visual clear (VC) zone. Used with interchange/ramp entry models, this refers to the zone that provides a buffer between the driver and the end of the acceleration lane, where the driver can either merge onto the freeway in a forced maneuver or abort the merge and begin to decelerate at a reasonable rate.

Yield line. A pavement marking used to mark the point of entry from an approach into the circulatory roadway of a roundabout, generally marked along the inscribed circle. Entering vehicles must yield to any circulating traffic coming from the left before crossing this line into the circulating roadway.

Zebra crossing. A crossing marked by transverse white stripes where vehicles are required to yield to pedestrians.

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This RD&T technical report provides full details of an experiment, its results, data collected from analysis, or findings and recommendations.