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Rural expressway intersection safety treatment evaluations

Joshua Lee Hochstein
Iowa State University

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by

Joshua Lee Hochstein

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Program of Study Committee:
Tom Maze, Co-Major Professor
Reginald R. Souleyrette, Co-Major Professor
Alicia L. Carriquiry
Tom Stout

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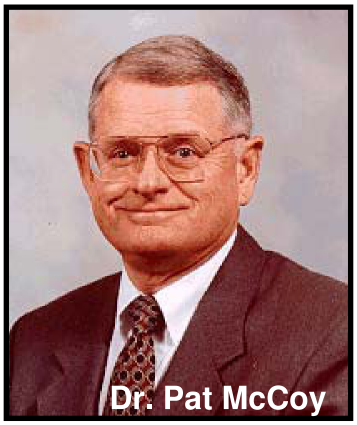
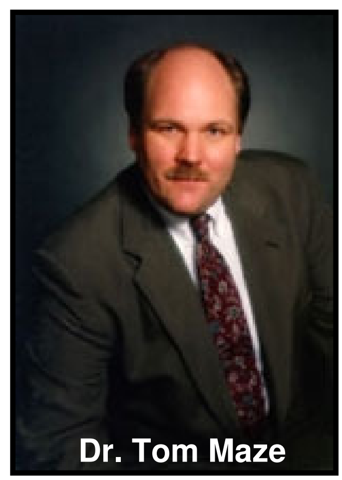
DEDICATION

This thesis is dedicated to my family (Leon, Mary, Justin, Sarah, and Nick) for all of their love and support throughout my life. Without them, this thesis would not have been possible. This thesis is also dedicated in memory of Dr. Pat McCoy and Dr. Tom Maze.



When I began my graduate studies at the University of Nebraska-Lincoln, Dr. McCoy was my major professor and mentor. He exhibited a strong work ethic and a passion for transportation research that was second to none. He jump started my interest in expressway intersection safety and authored a number of key research reports regarding this subject matter. He passed away due to lung cancer in October of 2002 at the age of 61, but never let his illness slow him down.

Similarly, if it weren't for Dr. Tom Maze, this thesis would not have been possible. He recruited me over to Iowa State University in the spring of 2005 after being awarded NCHRP 15-30, "Median Intersection Design for Rural High-Speed Divided Highways"; the project off which this thesis is based. Although Dr. Maze didn't know me well at the time, he wrote me an excellent letter of recommendation for the Eisenhower Fellowship which I was awarded that fall. Like Dr. McCoy, Dr. Maze was my major professor as well as a fabulous mentor and friend. He also exhibited a tireless work ethic, a great sense of humor, and a passion for conducting transportation research that made a difference in the world. Dr. Maze passed away due to heart failure on June 8, 2009 at the age of 57. I will always remember Dr. Maze for his honesty and for his ability to just be himself without worrying about what others may think. He said what was on his mind and didn't worry about the consequences. With that approach, you always knew where you stood with Dr. Maze.



Some have said that I must be cursed to have lost two advisors over my academic career; however, I don't feel that way. I have been extremely blessed for having the opportunity to learn from two of the greatest transportation researchers of the 20th century.

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LIST OF ABBREVIATIONS & ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ADT	average daily traffic
ASD	available sight distance
BBB	bouncing ball beacon
CALTRANS	California Department of Transportation
CCS	Collision Countermeasure System
CICAS-SSA	Cooperative Intersection Collision Avoidance Systems Stop Sign Assist
CIN	Commercial and Industrial Network
CSAH	County State Aid Highway
CTRE	Center for Transportation Research and Education
DHV	design hourly volume
DOT	Department of Transportation
FDOT	Florida Department of Transportation
FHWA	Federal Highway Administration
FIY	frequency per intersection per year
HAZ	hazardous approach zone
HCM	Highway Capacity Manual
hmev	hundred million entering vehicles
HSIS	Highway Safety Information System
IA-DOT	Iowa Department of Transportation
ICB	intersection control beacon
IDOT	Illinois Department of Transportation
IDS	Intersection Decision Support
ISD	intersection sight distance
ITE	Institute of Transportation Engineers
ITS	intelligent transportation systems
ITSDS	Iowa Traffic Safety Data Service
KDOT	Kansas Department of Transportation
LIDAR	Light Detection and Ranging
LOS	level-of-service
L-R	“left-right” offset T-intersection configuration
MAL	left-turn median acceleration lane
MD-313	Maryland State Highway 313
MDOT	Michigan Department of Transportation
mev	million entering vehicles
Mn/DOT	Minnesota Department of Transportation
MoDOT	Missouri Department of Transportation
mph	miles per hour
MSHA	Maryland State Highway Administration
MUTCD	Manual on Uniform Traffic Control Devices
mvm	million vehicle miles
NCDOT	North Carolina Department of Transportation

NCHRP	National Cooperative Highway Research Program
NCUTCD	National Committee on Uniform Traffic Control Devices
NDOR	Nebraska Department of Roads
NJDOT	New Jersey Department of Transportation
ODOT	Oregon Department of Transportation
OHPI	Office of Highway Policy Information
PDO	property damage only
PennDOT	Pennsylvania Department of Transportation
R-L	“right-left” offset T-intersection configuration
ROW	right-of-way
RSAR	road safety audit review
RTAL	right-turn acceleration lane
RTUT	Right-Turn U-Turn
SPF	safety performance function
SR	State Route
SSD	stopping sight distance
STA	State Transportation Agency
TRB	Transportation Research Board
TTA	time-to-arrival
TWSC	two-way stop control
USDOT	United States Department of Transportation
VDOT	Virginia Department of Transportation
VEWF	VEHICLES ENTERING WHEN FLASHING
vpd	vehicles per day
vph	vehicles per hour
WFET	WATCH FOR ENTERING TRAFFIC

ACKNOWLEDGMENTS

The research reported herein was performed as a portion of National Cooperative Highway Research Program (NCHRP) Project 15-30, "Median Intersection Design for Rural High-Speed Divided Highways" which will be published sometime in the near future. As such, I would like to thank the NCHRP for providing funding for this research, as well as the Iowa Department of Transportation and the Midwest Transportation Consortium (MTC).

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Dr. Tom Maze – Professor, CCEE
Dr. Reginald R. Souleyrette – Gerald & Audrey Olson Professor, CCEE
Dr. Alicia Carriquiry – Professor, Statistics
Dr. Tom Stout – Lecturer, CCEE

The late Dr. Tom Maze guided and directed the majority of this research prior to his passing. After his passing the other committee members deserve a lot of credit for continuing on during an extremely difficult time. I would especially like to thank Dr. Tom Stout for his contributions after being added to my committee extremely late in the game.

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ABSTRACT

A rural expressway is a high-speed, multi-lane, divided highway with partial access control which may consist of both at-grade intersections and grade separated interchanges. Many State Transportation Agencies (STAs) are converting rural two-lane undivided highways into expressways for improved safety and mobility; however, collisions at two-way stop-controlled (TWSC) intersections (particularly far-side right-angle crashes) on rural expressways are reducing the safety benefits that should be achieved through conversion. When the safety performance of these intersections begins to deteriorate, the improvement path typically begins with the application of several signing, marking, or lighting improvements, followed by signalization, and ultimately grade separation. Because signals hamper the mobility expressways are meant to provide and because interchanges are not economically feasible at all problematic intersections, there is a need for more design options at TWSC rural expressway intersections.

Some STAs have experimented with innovative rural expressway intersection safety treatments to avoid signalization and grade separation; however, little is known about the safety effects of these designs. Therefore, the objective of this research was to document their experience with these treatments and to conduct naïve before-after safety evaluations where possible. The ten "case studies" included within this thesis investigate J-turn intersections, offset T-intersections, jughandle intersections, Intersection Decision Support (IDS) technology, static roadside markers, left-turn median acceleration lanes (MALs), offset right-turn lanes, offset left-turn lanes, enhanced intersection guide signing, and dynamic advance intersection warning systems. These case studies help to begin to understand the safety improvement potential of these countermeasures and start to set the stage for the development of a richer set of design options at TWSC rural expressway intersections.

EXECUTIVE SUMMARY

Median-separated highways provide distinct benefits over undivided roadways (two-lane or multi-lane roads without medians). Medians separate opposing traffic, provide a recovery area for out-of-control vehicles, provide a stopping area in case of emergencies, allow space for speed change and storage lanes, and provide width for future lanes. In addition, rural multi-lane divided highways with partial or no access control (expressways) and low access densities provide safety performance and travel time benefits nearly equal to rural Interstates at a lower cost due to the fact that expressways can be built without purchasing full access rights and without constructing as many overhead bridges and interchanges.

Because of the expected safety/mobility benefits and lower costs of rural expressways (as compared to freeways), several states have built or are building expressway networks and plan to add additional miles to their systems. Most additions involve twinning an existing undivided two-lane highway, but in some cases, the expansion may involve the construction of a new corridor on separate right-of-way (bypasses) or other alignment improvements (i.e., curve flattening or realigning to be more consistent with the natural topography). However, several State Transportation Agencies (STAs) have seen the expected safety benefits of expressways (i.e., reductions in head-on and opposite direction sideswipe collisions) diminished by increased at-grade intersection collisions and increased intersection crash severity. Research has shown that the percentage of total expressway crashes which occur at two-way stop-controlled (TWSC) intersections increases as the mainline traffic volumes increase and that all intersection crashes increase and become more severe as minor roadway volumes increase.

The majority of collisions at TWSC expressway intersections tend to be right-angle crashes. The most problematic of these (with respect to frequency and severity) tend to be those occurring in the far-side intersection (i.e., after the minor road driver has traveled through the median). An initial response to this type of crash is to assume that the minor road driver did not recognize the intersection and ran the stop sign. However, examination of crash records in many states have shown that this is very infrequently the cause of expressway intersection crashes. More commonly, it has been found that minor road drivers

do stop, but then fail to select a safe gap in the expressway traffic stream (i.e., they misjudge the time-to-arrival of expressway vehicles). After addressing insufficient sight distance by clearing the roadside, the traditional approach to addressing safety problems at expressway intersections is to improve the visibility of traffic control devices, implement traffic signal control, and eventually construct an overpass or an interchange. However, traffic signals do not always improve safety (they may only change the crash type distribution and not the severity) and they drastically reduce the mobility expressways are meant to provide. In general, traffic signals in rural areas are discouraged for several reasons including violation of driver expectations and difficulty in powering and maintaining signals in remote locations. The final alternative is to build an interchange at the intersection. The construction of an interchange reduces the cost advantage of building an expressway as compared to building a freeway and the mix of at-grade intersections and interchanges tends to confuse driver expectations.

Therefore, the purpose of this project was to investigate alternative safety treatments at rural expressway intersections, to identify their relative effectiveness (if data were available), and to report any experiential information from those agencies who have tried the alternative. Although the traditional safety improvement path is from stop control to signal control to interchange construction, there are a myriad of non-traditional improvements which can be deployed to improve safety at a lower cost. These treatments can be categorized into three fundamental types (conflict point management techniques, gap selection aids, and intersection recognition devices) which are described within this thesis and have been shown (but not proven) to improve safety. A total of ten case studies of countermeasures from these categories were conducted. In almost all cases, the implementing agencies did not provide data that could be used for a rigorous statistical analysis, but in some cases, sufficient data were available to allow for naïve before and after crash comparisons. The case studies, discussed in Chapter 3, include the following rural expressway intersection safety treatments: J-turn intersections, offset T-intersections, jughandle intersections, Intersection Decision Support (IDS) technology, static roadside markers, left-turn median acceleration lanes (MALs), offset right-turn lanes, offset left-turn lanes, freeway-style advance intersection guide signing, and advance intersection warning systems.

The most important recommendation of this study is to continue developing the expressway intersection safety countermeasure matrix by further and more rigorously evaluating the safety effectiveness and volume thresholds for the different rural expressway intersection improvements. A national test protocol could be developed and followed as states deploy these improvements. For example, several states in the Midwest are planning to build J-turn intersections. If left to select their own evaluation protocols, inconsistencies in the analysis procedures will result making it difficult to perform rigorous statistical analyses and identify the safety improvement this countermeasure truly offers.

DISCLAIMER

This document was used in partial fulfillment of the requirements set forth by Iowa State University for the degree of Master of Science. The statistical analyses and resulting conclusions made in this thesis regarding “treatment effectiveness” are interim steps for a final report and should not reflect the final and/or current views of the Iowa Department of Transportation (Iowa DOT), The Center for Transportation Research and Education (CTRE), or Iowa State University (ISU). Further research involving the effectiveness of various rural expressway intersection safety treatments performed by the Iowa DOT and/or CTRE may alter the results reported herein.

CHAPTER 1: INTRODUCTION & RESEARCH APPROACH

1.1 BACKGROUND

A rural expressway is a high-speed (≥ 50 miles per hour (mph)), multi-lane, divided highway with partial access control. Although design policies vary from state to state, rural expressways are generally a hybrid design between a freeway and a conventional two-lane rural arterial roadway. Like freeways, rural expressways are typically four-lane divided facilities (i.e., two lanes in each direction separated by a wide, depressed, turf median) which may have grade separations and interchanges. However, expressway interchanges are generally limited to locations where the additional expenditure can be justified (i.e., at junctions with major highways, along bypasses, or at intersections with a historically disproportionate rate of serious crashes) because, like a conventional two-lane undivided rural arterial, expressways have partial access control allowing at-grade intersections and limited driveway access with the potential for signalization. Therefore, most intersections are at-grade. The typical rural expressway at-grade intersection, as shown in Figure 1, is two-way stop controlled (TWSC) with the stop control on the minor (usually two-lane) roadway. This traditional rural expressway intersection design has 42 conflict points as shown in Figure 2.

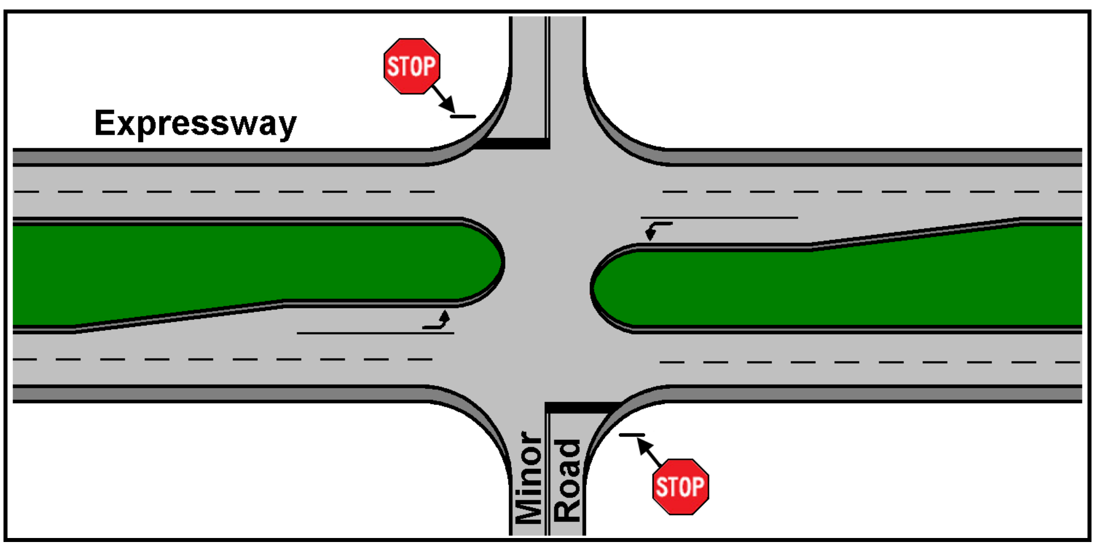
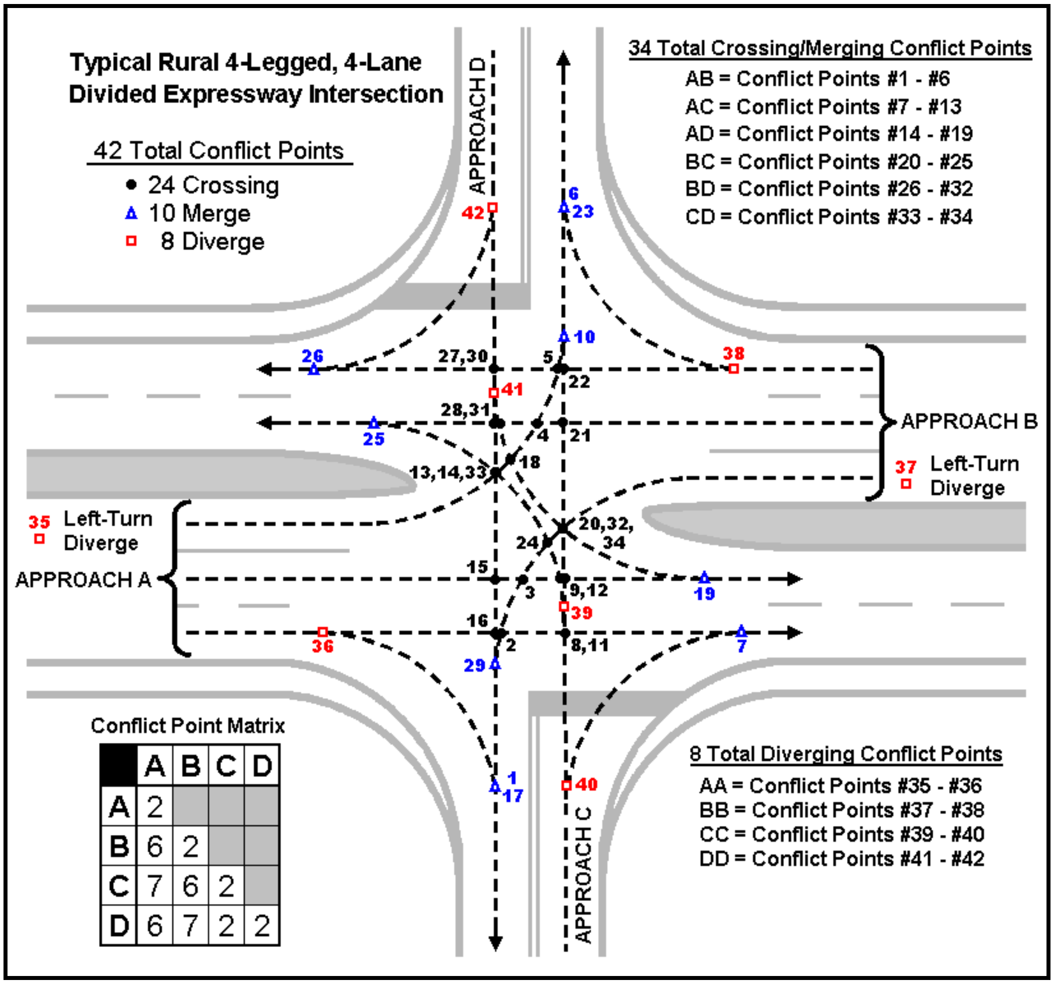


FIGURE 1. Typical rural expressway at-grade intersection

State Transportation Agencies (STAs) have been constructing and operating rural expressways since the early 1950’s in order to provide many of the safety, mobility, travel efficiency, and economic benefits of freeways at a far lower cost (*1*). Expressways are less expensive to build because they don't require the acquisition of as much right-of-way (ROW) or the construction of as many overhead bridges/interchanges and miles of frontage roads necessary to meet the freeway definition. Full access control and the associated grade separations can easily result in freeway construction costing double that of a comparable expressway corridor. Additionally, depending on state design standards, expressways may be designed using a less expensive cross-section (i.e., narrower medians and shoulders) (*2*). As a result, converting undivided rural two-lane highways into expressways has become a popular highway improvement used by many STAs.



	A	B	C	D
A	2			
B	6	2		
C	7	6	2	
D	6	7	2	2

FIGURE 2. Conflict point diagram for typical rural expressway intersection

By providing an extra lane of travel in each direction and a physical separation between opposing traffic flow, expressways make passing easier and drastically reduce the likelihood of dangerous head-on and opposite-direction sideswipe collisions experienced on rural two-lane highways. In addition, medians minimize headlight glare and provide: 1) a recovery area for out-of-control vehicles, 2) a stopping area in case of emergencies, 3) space for speed-change lanes, and 4) storage areas for left-turning and U-turning traffic (*3*). Minnesota Department of Transportation (Mn/DOT) data have shown that crash rates and severities on rural expressways are indeed lower than on rural two-lane highways: 0.9 crashes per million vehicle miles (mvm) with a fatality rate of 1.2 deaths per 100 mvm as compared to 1.0 and 1.6, respectively (*4*). Additionally, when access densities are low (≤ 5 access points per mile), crash rates on rural expressways drop to a level similar to that on rural freeways: 0.62 crashes per mvm on expressways compared with 0.60, respectively (*4*).

Besides these safety benefits, the popularity of conversion is also due to the fact that the high design speed and multi-lane cross-section enable expressways to operate, between intersections, with a capacity approaching that of a freeway; accordingly, expressways are considered more reliable facilities than conventional rural arterials. Consequently, expressways attract more trips, especially those made by the freight industry, and are viewed as an essential component for communities seeking to attract industry and economic development. Therefore, because expressways provide many of the safety, mobility, and economic benefits of freeways at a lower cost, expressways have become the fastest growing segment of the nation's rural highway system (*2*).

The Federal Highway Administration (FHWA) Office of Highway Policy Information's (OHPI's) Annual "Highway Statistics" Reports (*5*) were used to estimate the total rural expressway mileage in the U.S. on a state-by-state basis in 1995 and 2005. The data obtained is presented in Table 1. Over this decade, rural expressway mileage increased nationally by 2,400 miles or 16 percent with 13 states having added over 100 miles to their rural expressway systems. The majority of this expansion has been done through the conversion of undivided two-lane rural highways into expressways and this trend is likely to continue as a 2004 survey of STAs indicated that 26 of the 28 responding agencies had plans to expand their state expressway systems over the next ten years (*2*). Some states have explicit, strategic programs for upgrading two-lane highways on uniquely identified networks to multi-lane divided standards. Examples include the Nebraska State Expressway System and the Iowa Commercial and Industrial Network (CIN). Other agencies, like the Missouri

Department of Transportation (MoDOT), have simply made a commitment to upgrade rural two-lane highways to expressways in order to provide better connectivity between regional centers within the state (*6*).

TABLE 1. Estimated Rural Expressway Mileage by State, 1995 & 2005 (*5*)

STATE	1995 TOTAL MILES*	2005 TOTAL MILES*	MILES CHANGE (1995-2005)	% CHANGE	MILES CHANGE RANK	% CHANGE RANK	2005 MILES RANK
New Mexico	346	830	484	139.88	1	5	3
Iowa	194	493	299	154.12	2	4	16
Nebraska	46	320	274	595.65	3	1	23
Tennessee	286	527	241	84.27	4	7	15
South Dakota	196	395	199	101.53	5	6	20
West Virginia	243	429	186	76.54	6	8	17
Virginia	931	1097	166	17.83	7	17	2
North Carolina	442	580	138	31.22	8	14	12
Arizona	65	194	129	198.46	9	3	29
Wisconsin	227	348	121	53.30	10	10	21
Kentucky	214	329	115	53.74	11	9	22
Oklahoma	430	538	108	25.12	12	15	14
Alabama	592	694	102	17.23	13	18	7
Texas	1978	2071	93	4.70	14	23	1
Illinois	204	292	88	43.14	15	11	24
Ohio	678	751	73	10.77	16	19	5
Minnesota	618	679	61	9.87	17	20	8
Kansas	115	151	36	31.30	18	13	31
California	552	587	35	6.34	20	21	11
Florida	637	672	35	5.49	21	22	9
Washington	188	223	35	18.62	19	16	27
Mississippi	781	813	32	4.10	22	26	4
Arkansas	73	104	31	42.47	23	12	34
Missouri	704	733	29	4.12	24	25	6
Georgia	572	598	26	4.55	25	24	10
Indiana	557	575	18	3.23	26	27	13
Wyoming	2	8	6	300.00	27	2	42
Alaska	5	5	0	0.00	28	28	43
Maine	0	0	0	0.00	29	29	47
Connecticut	1	0	-1	-100.00	31	48	48
Utah	55	54	-1	-1.82	30	30	37
Hawaii	6	3	-3	-50.00	32	45	44
Nevada	46	41	-5	-10.87	33	31	39
Vermont	25	20	-5	-20.00	34	37	40
Massachusetts	7	0	-7	-100.00	35	49	49
New Hampshire	10	0	-10	-100.00	36	50	50
Montana	25	13	-12	-48.00	37	44	41
Rhode Island	16	3	-13	-81.25	38	46	45
Idaho	65	50	-15	-23.08	39	39	38
Delaware	127	106	-21	-16.54	40	34	33
New York	222	185	-37	-16.67	41	35	30
Michigan	110	64	-46	-41.82	42	43	36
Oregon	125	78	-47	-37.60	43	42	35
North Dakota	464	405	-59	-12.72	44	32	19
Maryland	360	290	-70	-19.44	45	36	25
Louisiana	317	246	-71	-22.40	46	38	26
South Carolina	493	421	-72	-14.60	47	33	18
Colorado	223	149	-74	-33.18	48	41	32
Pennsylvania	305	212	-93	-30.49	49	40	28
New Jersey	98	3	-95	-96.94	50	47	46
U.S. TOTALS	**14976**	**17379**	**2403**	**16.05**			

* Non-Interstate, NHS, Rural Divided Highways (≥ 4 Lanes with Partial or No Access Control)

1.2 PROBLEM STATEMENT

The Nebraska Department of Roads (NDOR) is also in the process of building an extensive program of rural expressways. This fact is evident in Table 1. Between 1995 and 2005, Nebraska ranked third in rural expressway miles added and concurrently had the largest percentage increase in total rural expressway mileage. As part of a 1988 NDOR Needs Study, engineers used socioeconomic data to designate an expanded State Expressway System. The rural two-lane highways selected for upgrade were chosen: 1) to connect urban centers with a population of 15,000 or more to each other and to I-80, 2) to add routes with an average daily traffic (ADT) volume of 500 or more heavy commercial vehicles, and 3) to add additional segments for continuity. When this program is complete, the Nebraska Expressway System will be approximately 600 miles in length (*7*).

In the midst of this program, NDOR wanted to find out if upgrading these two-lane highways to expressway standards was improving safety as expected. Therefore, in 2000, NDOR's Highway Safety Division compared crash rates for 111 miles of rural expressways (i.e., sections with less than 8,000 vehicles per day (vpd)) with 324 miles of rural two-lane highways planned for expressway conversion (*8*). The results of this study are presented in Table 2. For each roadway segment included in the analysis, at least two years of crash data were used in the crash rate calculations; three years of data were used where possible. As Table 2 shows, crash rates for head-on and opposite-direction sideswipe collisions were substantially lower on rural expressways as anticipated. Unfortunately, this level of improvement did not extend to all crash types. The major concern is the 71 percent higher right-angle crash rate on expressways. While other intersection-related, multi-vehicle crash types (i.e., rear-end/left-turn) had lower crash rates on expressways, the overall intersection-related crash rate was two percent higher on expressways due to the elevated right-angle crash experience. In addition, the overall crash rate was five percent higher on expressways.

As a result of these findings, NDOR concluded that right-angle intersection collisions on their existing rural expressways seem to be negating the safety benefits that should be derived from converting rural two-lane highways into expressways (*8*). Similarly, Preston et al. (*4*) reviewed three years (2000 to 2002) of rural TWSC intersection crash data for Mn/DOT and found that rural expressway intersections have a greater proportion of right-angle collisions than intersections on rural two-lane highways; 36 to 26 percent, respectively. Right-angle crashes are potentially more severe on expressways due to the higher speed of mainline traffic and are consequently, cause for concern; especially as many STAs plan to continue converting two-lane highways into expressways.

TABLE 2. Nebraska Crash Experience: 2-Lane Highways vs. 4-Lane Expressways (*8*)

	CRASH RATES (crashes/million vehicle miles)		
CRASH TYPE	324 MI OF RURAL 2-LANE HWYS PLANNED FOR EXPRWY CONVERSION	111 MI OF RURAL 4-LANE DIVIDED EXPRWYS (ADT < 8000 vpd)	PERCENT DIFF.
All Crashes	**0.942**	**0.991**	**+ 5.2**
Fatal	0.022	0.015	-31.8
Injury	0.324	0.309	-4.6
Property Damage Only	0.596	0.668	+ 12.1
Single-Vehicle Crashes	**0.556**	**0.661**	**+ 18.9**
Run-Off-Road	0.213	0.247	+ 16.0
Animal	0.309	0.381	+ 23.3
Multiple-Vehicle Crashes	**0.386**	**0.330**	**-14.5**
Sideswipe (Opposite)	0.067	0.005	-92.5
Head-On	0.012	0.004	-66.7
Rear-End	0.131	0.093	-29.0
Sideswipe (Same)	0.064	0.051	-20.3
Left-Turn	0.026	0.025	-3.8
Right-Angle	0.087	0.149	+ 71.3
Intersection-Related Crashes	**0.231**	**0.236**	**+2.2**

The right-angle crash problem at rural expressway intersections is not specific to Nebraska and Minnesota. Utah and Iowa data have shown similar trends. Minnesota and Utah data were presented in National Cooperative Highway Research Program (NCHRP) Report 375 (*9*). This data showed that 42 percent of all rural divided highway crashes in Minnesota (34 percent in Utah) were intersection-related and 57 percent of those collisions (69 percent in Utah) were right-angle or turning crashes. A more recent study showed that 52 percent of all rural expressway intersection collisions in Iowa were of the right-angle variety (*2*). Therefore, these statistics illustrate that right-angle intersection collisions constitute an important issue in rural expressway safety management.

During the 1990's, the Iowa Department of Transportation (IA-DOT) upgraded a number of routes on the Iowa CIN to expressway standards. The Iowa CIN is comprised of 2,275 miles of primary highways identified by the state legislature to enhance opportunities for the development and diversification of the state's economy (*10*). As shown in Table 1, between 1995 and 2005, Iowa ranked second in rural expressway miles added and had the fourth highest percentage increase in total rural expressway mileage. Current IA-DOT long-range plans call for an additional 355 miles of the CIN to be constructed to four-lane divided standards (84 miles of which are included in the 2009-2013 Iowa Transportation Improvement Program). Under the best circumstances, if the conditions that result in the most problematic intersections are known during the corridor planning stage of an expressway's development, those conditions can be prevented by highway planners and designers. For existing facilities, locations with problematic conditions can be identified and traffic safety engineers can proactively program the appropriate improvements.

Therefore, in order to better understand the safety performance of TWSC expressway intersections, a number of studies over the years have examined the relationship between the frequency of crashes and traffic volume through the development of safety performance functions (SPFs). The first to do so was McDonald (*1*) in 1953. He examined 150 unsignalized, divided highway intersections (both 3 and 4-legged) in rural California. Then, in 1964, Priest (*11*) studied 316 divided highway intersections in Ohio (the author did not explicitly state the area type, traffic control type, or the number of intersection legs for the sample intersections). Next, in 1992, Bonneson and McCoy (*12*) investigated 125 TWSC intersections in rural Minnesota using an FHWA Highway Safety Information System (HSIS) data subset (apparently only 17 of these intersections were on divided highways). In 1995, Harwood et al. (*9*) developed separate SPFs for 153 TWSC and 157 unsignalized, three-legged divided highway intersections in rural California. Finally, in 2004, Maze et al. (*2*) developed a SPF using 644 TWSC expressway intersections in rural Iowa. The relationships developed as a result of these studies are summarized in Table 3 and Figure 3. Four of these five studies concluded that crash frequency is more sensitive to changes in the minor roadway volume than to changes in the divided highway volume as indicated by the larger values of the minor roadway volume coefficients (*1, 2, 11, 12*).

Maze et al. (*2, 13, 14*) examined this phenomenon in closer detail for the IA-DOT by examining how crash rates, crash severity, and crash types are affected by increasing traffic volumes and other various site conditions. Maze et al. (*13*) found that as expressway volumes increase, crashes more commonly occur at intersections (as opposed to between them). These results are presented in Figure 4. In addition, Maze et al. (*2*) found that intersection crash rates and severity are highly dependent on the volume of traffic entering the intersection from the minor road. As minor road traffic volumes increase, the intersection crash rates increase and the crashes become more severe as shown in Figure 5. Furthermore, as entering minor road volumes increase, the distribution of intersection crash types changes as shown in Figure 6, with a higher proportion of right-angle crashes occurring. Because right-angle crashes are more likely to be severe, increasing minor road volume results in

TABLE 3. Summary of SPFs Developed for Divided Highway Intersections

REFERENCE	SAMPLE	EQUATION	R^2
McDonald, 1953 (*1*)	150 unsignalized divided highway intersections (both 3 and 4-leg) in rural California	$N = 0.000783(V_{MAJ})^{0.455}(V_{MIN})^{0.633}$	Not Given
Priest, 1964 (*11*)	316 divided highway intersections in Ohio (area type, traffic control, and number of intersection legs not specified)	Graphical expression as presented in Figure 3	Not Given
Bonneson & McCoy, 1992 (*12*)	125 TWSC intersections in rural Minnesota (108 on two-lane major road, 17 on four-lane divided highways)	$N = 0.6503\left(\frac{V_{MAJ}}{1000}\right)^{0.2925}\left(\frac{V_{MIN}}{1000}\right)^{0.7911}$	Not Applicable
Harwood et al., 1995 (*9*)	153 TWSC divided highway intersections in rural California	$Y = e^{-20.498}(V_{MAJ})^{0.672}(V_{MIN})^{0.575} * e^{-0.013X_1+0.961X_2+0.328X_3-0.354X_4+0.317X_5+0.396X_6-0.233X_7}$	0.3914
	157 unsignalized, three-legged, divided highway intersections in rural California	$Y = e^{-12.677}(V_{MAJ})^{1.110}(V_{MIN})^{0.324} * e^{0.838X_3+0.732X_4-0.399X_7+0.478X_8}$	0.3454
Maze et al., 2004 (*2*)	644 TWSC rural expressway intersections in Iowa	$N = e^{0.02278+(0.00005*V_{MAJ})+(0.00042*V_{MIN})}$	0.381

N = Expected number of crashes per year
Y = Expected number of multiple-vehicle crashes over a 3 year period
V_{MAJ} = ADT (vpd) entering from the major road (divided highway)
V_{MIN} = ADT (vpd) entering from the minor road (crossroad)
X_1 = Median width (ft)
X_2 = Average lane width on major road (ft)
X_3 = 1 if intersection lighting is present, 0 otherwise
X_4 = 1 if left-turn channelization is present on the major road, 0 if not
X_5 = 1 if functional class of major road is 4 or 5, 0 if functional class is 1, 2, or 3
X_6 = 1 if major road has 4 lanes in both directions combined, 0 if less than 4 lanes combined
X_7 = 1 if terrain is rolling or mountainous, 0 if terrain is flat
X_8 = 1 if partial access control on major road, 0 if no access control

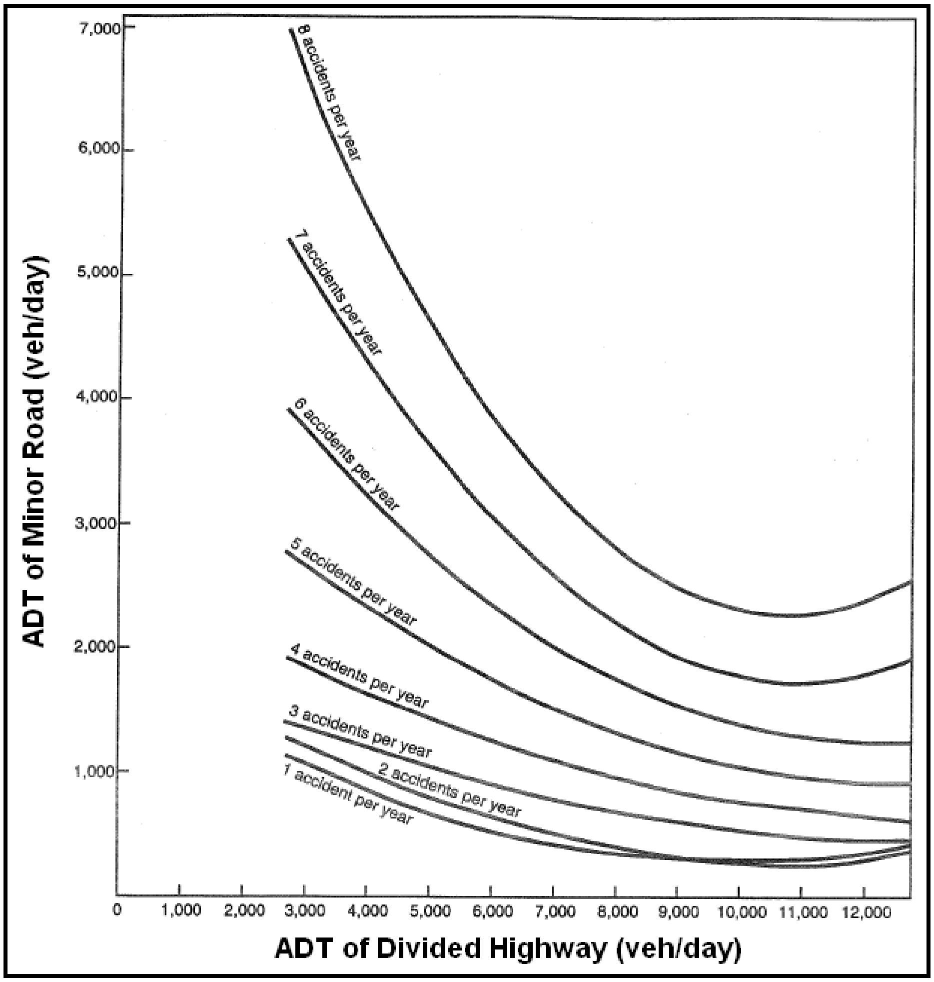


FIGURE 3. Crash frequency & volume relationship developed by Priest (11)

increased crash severity as illustrated in Figure 5. When similar bar charts were developed and stratified by entering expressway volumes, intersection crash rates, severity rates, and crash type distribution did not tend to change with increasing expressway volumes in Iowa (*2*). However, in Minnesota, where some rural expressways have become heavy commuter routes into the Twin Cities Metropolitan Area, thus experiencing much larger volumes than in Iowa, Preston et al. (*4*) observed that right-angle crashes do become more prevalent with increasing expressway volumes as shown in Figure 7.

From the set of 644 TWSC rural expressway intersections in Iowa, Burchett and Maze (*14*) identified the 100 highest-severity and the 100 lowest-severity intersections (in terms of severity index rate as described in Figure 5) from among the 327 intersections which had experienced at least one crash during the study period (1996-2000) for the purpose of conducting a comparative statistical analysis. In their analysis of land use, they found that 75 percent of the low-severity intersections were bordered by agricultural land, whereas 86 percent of the high-severity intersections were bordered by residential or commercial land-use. Further investigation revealed that the fatality rate for intersections located adjacent to residential land use was 79 percent and 32 percent greater than for intersections located adjacent to commercial and agricultural land use, respectively. Because residential

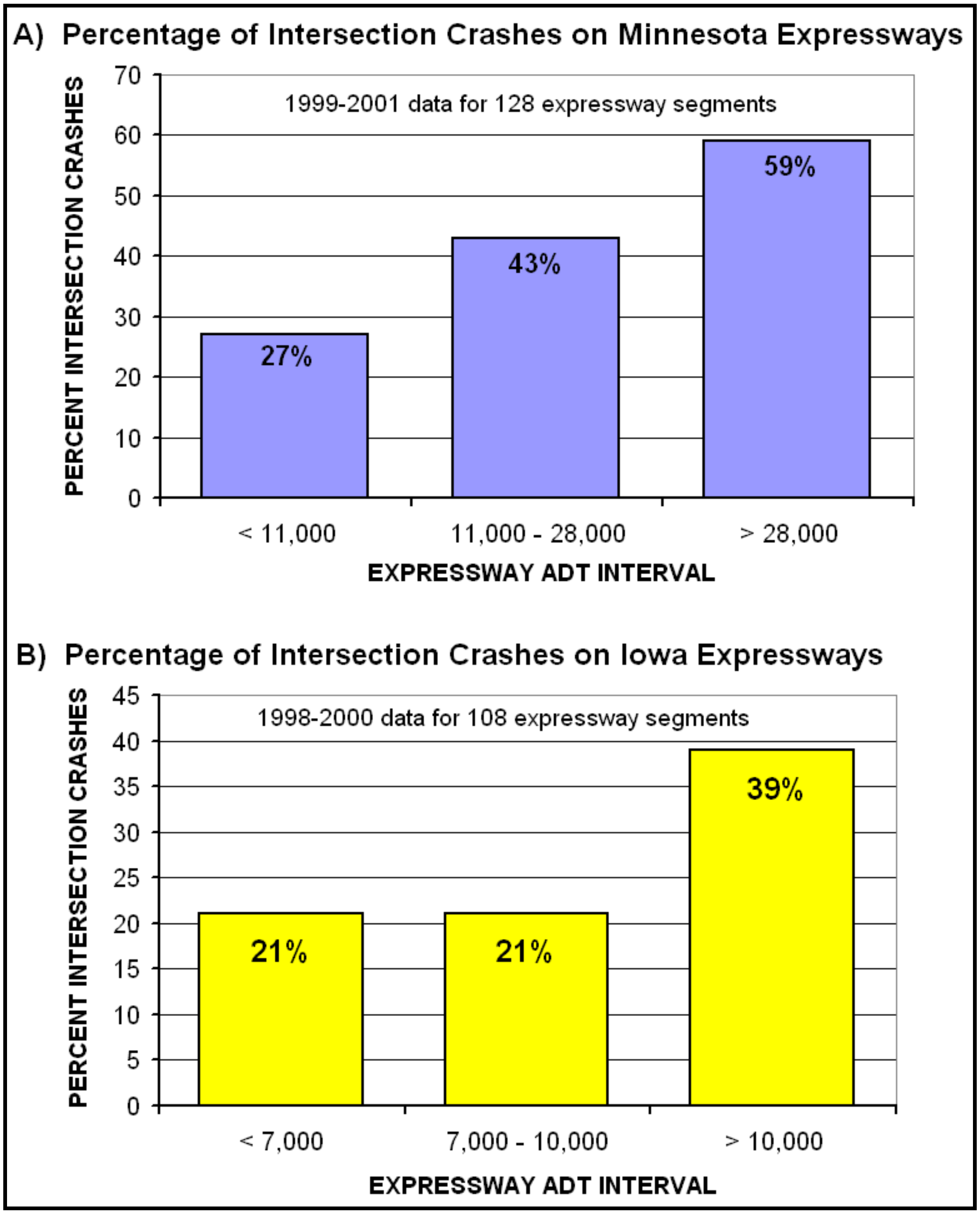


FIGURE 4. Percent intersection crashes on expressways by expressway volume (13)

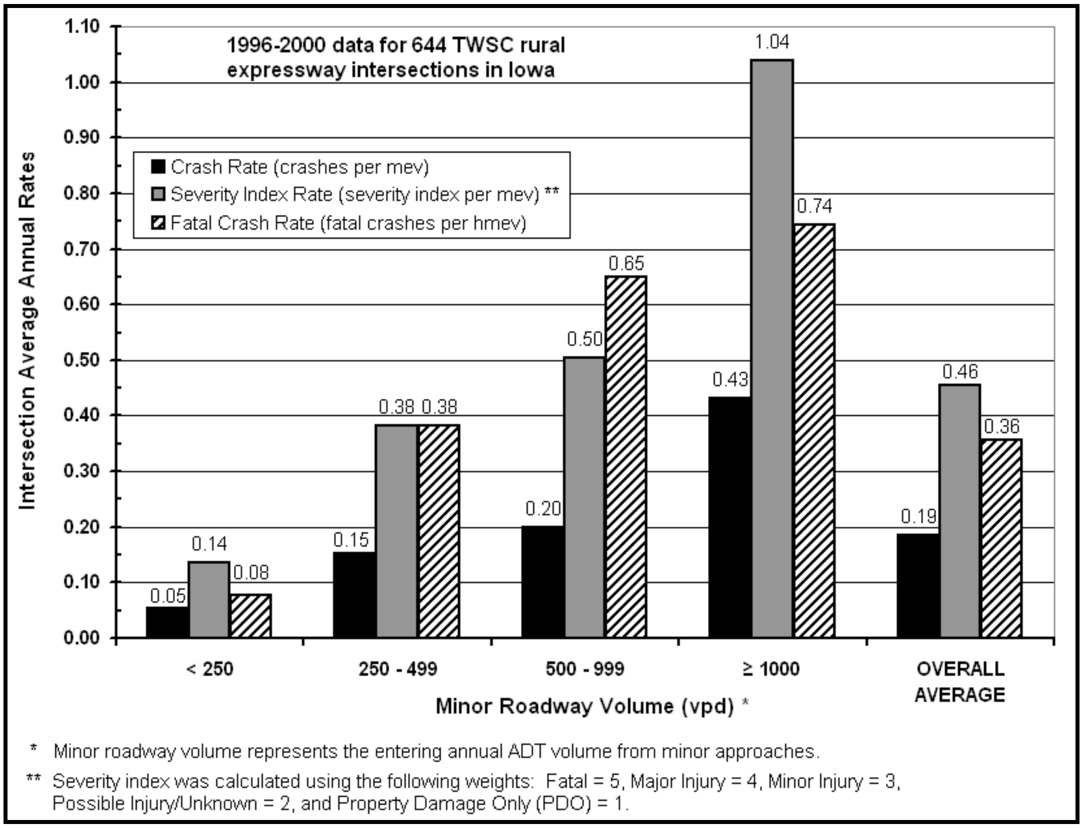


FIGURE 5. Intersection crash rates & severity by minor road entering volume (2)

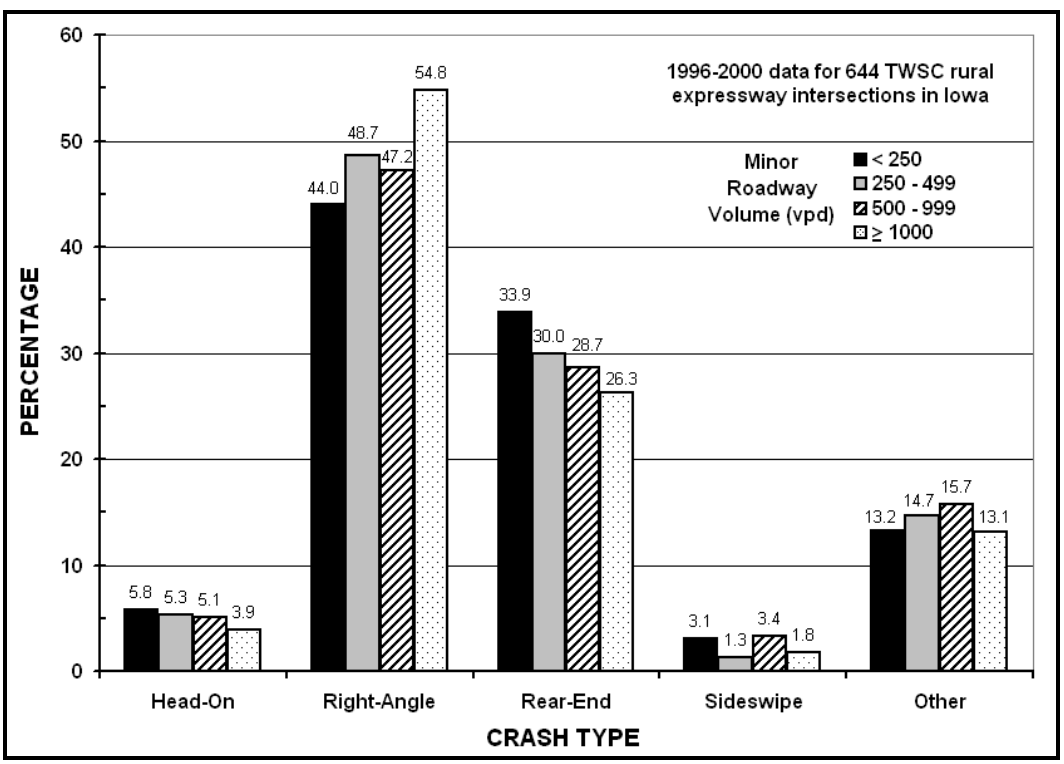


FIGURE 6. Intersection crash type distribution by minor road entering volume (2)

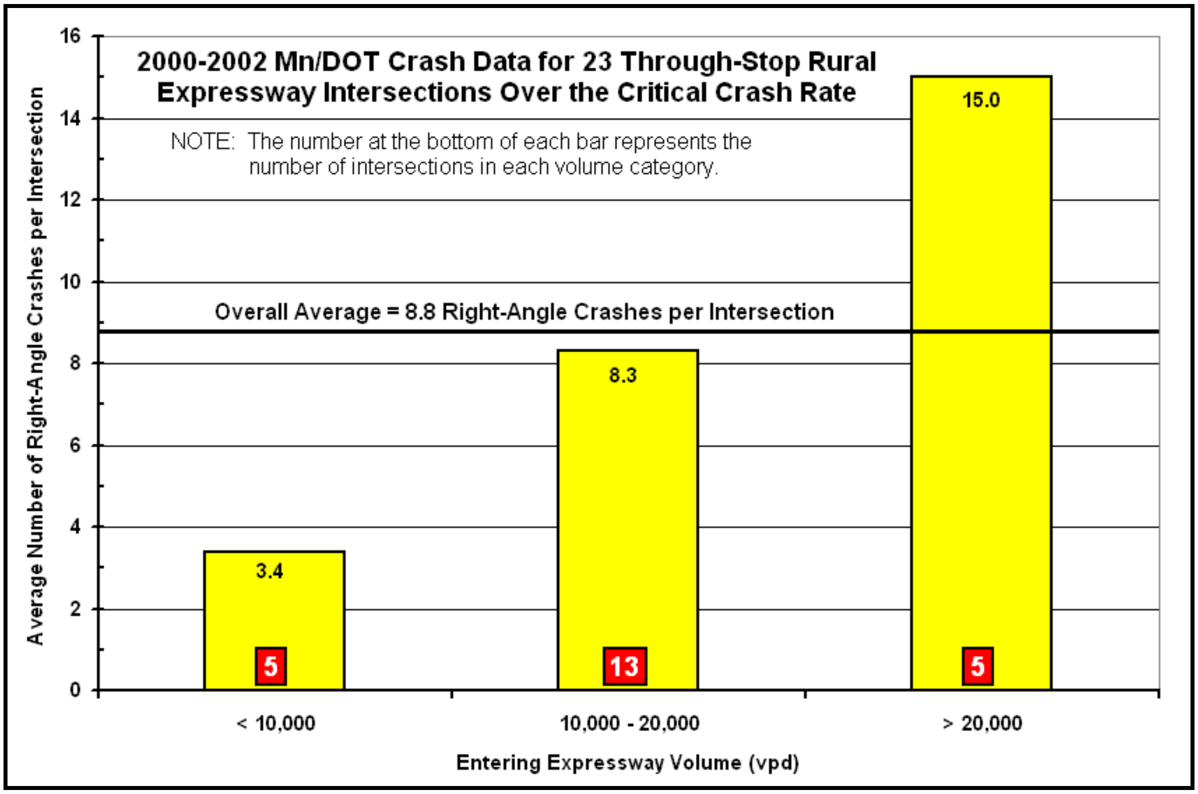


FIGURE 7. Effect of expressway volumes on right-angle crashes in Minnesota (4)

development serves as a proxy for peak hour volumes as commuters travel to and from work, Burchett and Maze (*14*) obtained 24-hour traffic counts for a sample of 30 intersections with the highest crash severity index rates in order to examine the impact of hourly peaking on safety performance. After determining the peak hours for each intersection, they found that the 30 highest severity intersections experienced extreme hourly peaking with 52 percent of the crashes occurring during the peak hours.

In addition, Maze et al. (*2*) and Preston et al. (*4*) both found that the distribution of crashes at TWSC rural expressway intersections with the worst safety performance tend to be heavily skewed toward right-angle crashes. Maze et al. found that the 10 highest crash severity intersections in Iowa (those where the actual severity index exceeded the expected index by the greatest amount) had 66.3 percent right-angle crashes as compared with 13.0 percent at the 10 lowest crash severity intersections (those where the expected severity index exceeded the actual index by the greatest amount). Similarly, Preston et al. observed 53 percent right-angle crashes at 23 rural expressway intersections in Minnesota where the crash rates exceeded the critical crash rate versus 36 percent right-angle crashes at 396 rural expressway intersections across the state. Furthermore, Burchett and Maze (*14*) found that almost 60 percent of the crashes occurring at the 100 highest-severity and the 100 lowest-severity intersections were of the right-angle variety when vertical curvature, horizontal curvature, or intersection skew were present versus a 52 percent statewide average.

In an effort to develop a better understanding of the causes of right-angle crashes at TWSC rural expressway intersections, Preston et al. (*4*) performed a detailed review of the crash reports at three of the intersections over the critical crash rate and found that: 1) 87 percent of the right-angle crashes were due to the inability of minor road drivers to recognize oncoming expressway traffic and/or select safe gaps in the expressway traffic stream, 2) 78 percent of the right-angle crashes were "far-side" collisions (i.e., right-angle crashes involving left-turning or crossing minor road vehicles which successfully cross the first (near-side) set of expressway lanes, but collide with expressway traffic in the second (far-side) set of lanes after traversing through the median (the concept of near and far-side collisions is illustrated in Figure 8)), and 3) intersection recognition (i.e., running of the stop sign) by drivers on the minor, stop-controlled approaches was not a contributing factor in any of the right-angle crashes at these intersections. Similarly, Burchett and Maze (*14*) found that the ratio of far-side to near-side collisions at 30 TWSC rural expressway intersections with the highest crash severity indices in Iowa was 62 percent to 38 percent. However, at 7 of these intersections where horizontal curves were present along the expressway, far-side and near-side collisions were nearly equally distributed at 51 and 49 percent, respectively. Therefore, horizontal curves on the mainline seem to create a unique hazard for minor road drivers attempting to select gaps at both the near and far-side intersections.

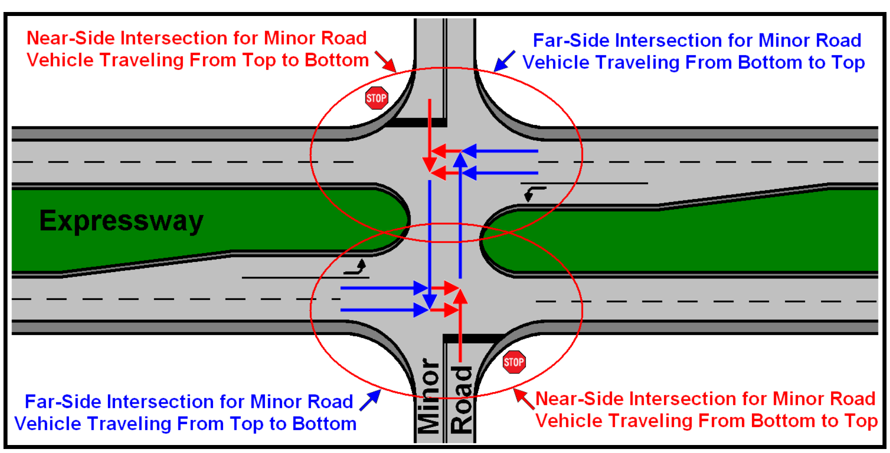


FIGURE 8. Far-side & near-side intersection definitions

From all of these observations, it appears that the primary safety issue at TWSC rural expressway intersections is right-angle collisions (far-side right-angle crashes in particular). The predominant cause of these crashes seems to be the inability of minor road drivers to judge the speed and distance (i.e., arrival time) of approaching expressway vehicles as they attempt to enter or cross the expressway. Preston et al. (*4*) speculated that some of these mistakes in judgment may occur because minor road drivers are using a "one-stage" gap selection process. A one-stage gap selection occurs when a minor road driver simultaneously tries to find an acceptable gap in expressway traffic coming from both the left and right, thereby attempting to cross or turn left in a single motion without stopping in the median to re-evaluate if the gap in traffic coming from the right is still adequate. On the other hand, "two-stage" gap selection is less complex and far less demanding on the minor road driver because it breaks down the crossing or left-turning process into four easier successive tasks. First, the minor road driver focuses only on finding a gap in expressway traffic coming from the left. Once the minor road driver has crossed the near-side expressway lanes, he or she then stops in the median and focuses on expressway traffic coming from the right. Finally, the minor road driver proceeds to enter or cross the far-side expressway lanes when an acceptable gap is available. Another possible contributing factor to minor road drivers misjudging approaching expressway vehicle speeds and distances may be the fact that many of these intersections are in rural areas where there are no large fixed objects which can be used as points of reference to help gage vehicle positioning.

Other intersection geometric design features (horizontal and vertical curvature on the expressway, intersection skew, median width, etc.) make the task of gap selection more difficult for the minor road driver. In addition, minor road driver age plays a role in their ability to safely navigate through a TWSC expressway intersection. Young drivers have more difficulty due to their inexperience and elderly drivers have more problems due to their naturally declining visual, motor, and cognitive processing capabilities. A survey of elderly drivers in West Virginia (*15*) indicated that more than half of the respondents had problems making turning or crossing maneuvers from the minor road at TWSC expressway intersections and their greatest difficulty was stated as judging the speed of oncoming vehicles. Maze et al. (*2*) found that the driver age distribution of those involved in injury and fatal crashes was similar for rural TWSC expressway intersections in Iowa versus all rural

TWSC intersections statewide; however, younger (< 25 years of age) and older (> 55 years of age) drivers were over-represented in right-angle crashes at rural expressway intersections.

The safety performance of conventional TWSC rural expressway intersections declines as entering traffic volumes increase, especially from the minor road or where minor roadway volumes are highly peaked. At the most problematic intersections, right-angle collisions are exacerbated as gap selection becomes more of an issue (see Figure 9). As expressway volumes increase, the number of safe gaps in the expressway traffic stream declines and right-angle crashes become more prevalent. As minor road volumes increase, there are more vehicles trying to use the same number of safe gaps, thus there is an increased probability for right-angle crashes to occur. Furthermore, when more traffic is present on the minor road approaches (i.e., congestion during peak hours) there may be more "peer" pressure on the lead minor road driver to select a gap that he/she would not normally accept, resulting in a higher number of unsafe gaps being selected.

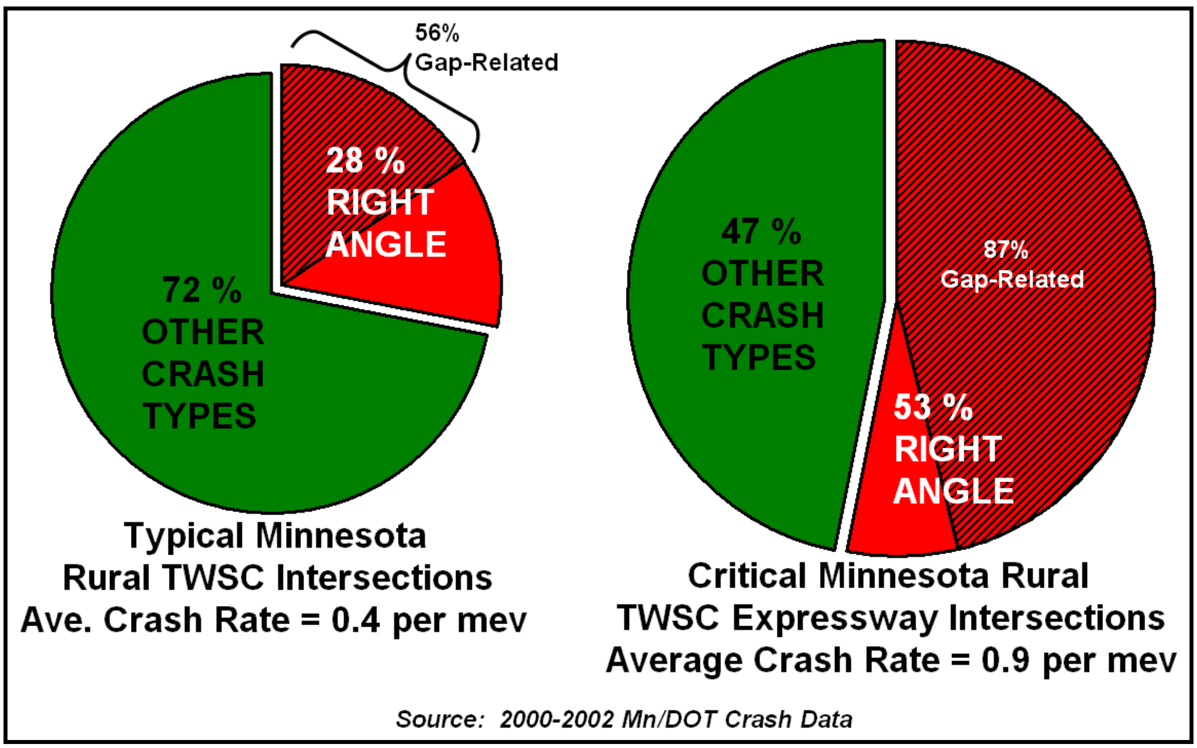


FIGURE 9. Crash causation comparison for rural TWSC intersections in Minnesota (4)

1.3 RESEARCH OBJECTIVES

As a result of the trend to convert rural two-lane undivided roadways into multi-lane divided highways, rural expressways are a rapidly growing component of the nation's transportation network. As these facilities experience growth in traffic, at-grade intersection collisions begin to reduce the safety benefits that should be achieved as a result of conversion, bringing into question the assumption that expressways are an effective alternative to full access-controlled facilities at high volumes. When the safety performance of these at-grade intersections begins to deteriorate, the traditional reactive approach taken by STAs is to consider improvements one intersection at a time. Often, the improvement path starts with the application of several signing, marking, and/or lighting improvements, followed by the implementation of traffic signals, and ultimately grade separation as shown in Figure 10. The problem with this method is that it is reactive to problems at intersections with historically poor safety records. Substantial time is required to fully determine that a safety problem exists, and then the design, construction, and/or implementation of a solution may take many years, during which time the problem may continue or worsen. For example, the environmental work, preliminary and final design, ROW acquisition, and construction of a rural interchange may be a five year project or more. Moreover, the high cost of constructing a grade separation or an interchange limits their use on expressways.

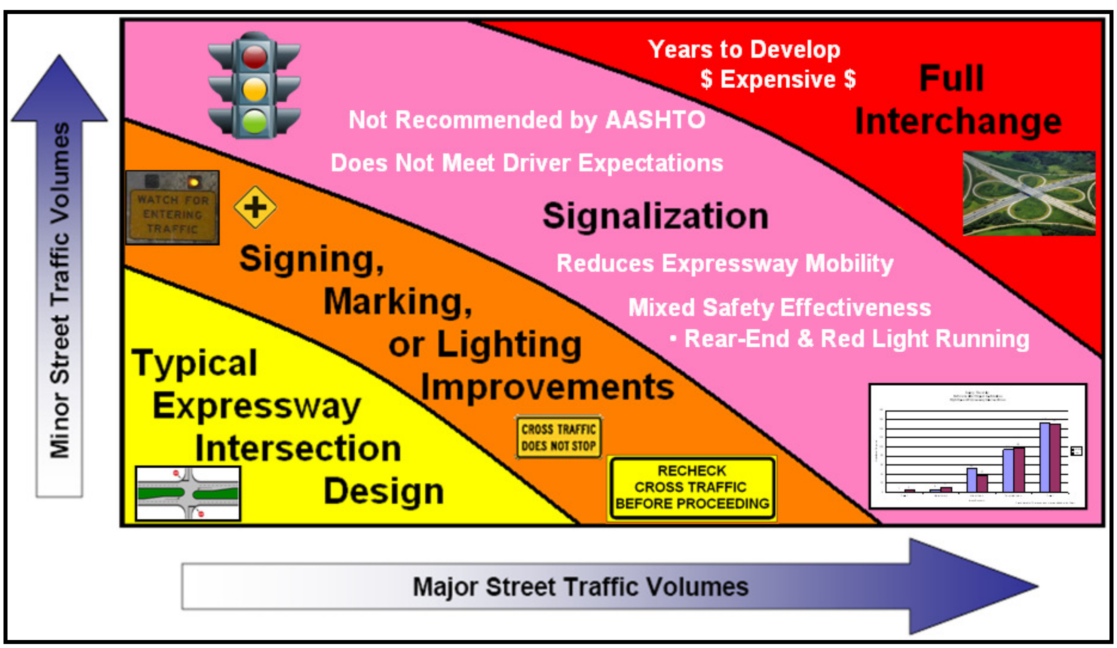


FIGURE 10. Traditional reactive TWSC rural expressway intersection treatment matrix

When an interchange is not economically justified, when the funding is not available, or while an interchange is being developed, the traditional interim corrective measure for a TWSC expressway intersection is signalization. However, rural expressway intersections often experience safety problems long before they meet traffic signal volume warrants. Furthermore, the *AASHTO Policy on Geometric Design of Highways and Streets* (a.k.a. the Green Book) (*3*) and *NCHRP 500, Volume 5: A Guide for Addressing Unsignalized Intersection Collisions* (*16*) guard against using signalization as a safety device and state that intersection control by traffic signals should be avoided whenever possible. On rural expressways in particular, signals are dangerously inconsistent with the expressway driver's expectation of a free flow roadway for high speed travel; thus creating high potential for rear-end crashes and red-light running. Research on the safety effects of signalizing divided highway intersections has shown mixed results (*17, 18, 19, 20, 21*). In general, signalization leads to an increase in crash rates with reduced severity due to a shift in crash types (*16*); however, large variability in the safety effectiveness of signalization at individual locations has been observed (*21*). In addition, Bonneson and McCoy (*12*) concluded that the costs of stopping expressway traffic are so high that a very heavy minor road demand must be present to economically justify installing a traffic signal, and when volumes reach those levels, a diamond interchange is a more economically viable alternative. Thus, some states have established policies that prohibit signalizing rural expressway intersections, preferring to design interchanges where large minor roadway traffic volume levels are anticipated. Of course, in the long-run, this practice will be expensive and impractical if widely applied. Therefore, designers must have other options besides signalization and grade separation at their disposal to address rural expressway intersection safety.

STAs have experimented with a wide range of intersection safety treatments at problematic rural expressway intersections to improve their safety performance while avoiding signalization and grade separation. A number of these strategies are identified and briefly described in *NCHRP 500, Volume 5* (*16*). Unfortunately, few states have adequately examined the safety effects of these treatments as most are listed as "tried" or "experimental" strategies as shown in Table 4. In order to determine the safety effectiveness of these alternatives and classify them as "proven", the "tried" and "experimental" strategies will need to be appropriately implemented and evaluated scientifically through rigorous before-after studies. However, some STAs are reluctant to implement and test these strategies because design guidance is lacking and is generally silent on their application.

TABLE 4. NCHRP 500, Vol. 5 (*16*) – Unsignalized Intersection Safety Strategies

Strategies Specific to Rural Expressway Intersections		
NO.	**STRATEGY DESCRIPTION**	**TYPE**
B5	Provide left-turn acceleration lanes at divided highway intersections (MALs)	Tried
B12	Restrict/eliminate turning maneuvers through channelization or closing median openings	Tried
B17	Use indirect left-turn treatments to minimize conflicts at divided highway intersections	Tried
C2	Clear sight triangles in the medians of divided highways near intersections	Tried
E7	Provide dashed markings (extended left edgelines) for major road continuity across the median opening at divided highway intersections	Tried
I2	Provide a double yellow centerline in the median opening of divided highway intersections	Tried

Strategies Which May Be Applied at Rural Expressway Intersections		
NO.	**STRATEGY DESCRIPTION**	**TYPE**
B1	Provide left-turn lanes at intersections	Proven
B2	Provide longer left-turn lanes at intersections	Tried
B3	Provide offset left-turn lanes at intersections	Tried
B6	Provide right-turn lanes at intersections	Proven
B7	Provide longer right-turn lanes at intersections	Tried
B8	Provide offset right-turn lanes at intersections	Tried
B9	Provide right-turn acceleration lanes at intersections	Tried
B10	Provide full width paved shoulders in intersection areas	Tried
B11	Restrict or eliminate turning maneuvers by signing	Tried
B13	Close or relocate “high-risk” intersections	Tried
B14	Convert four-legged intersections to two T-intersections	Tried
B15	Convert offset T-intersections to four-legged intersections	Tried
B16	Realign intersection approaches to reduce or eliminate intersection skew	Proven
C1	Clear sight triangles on stop or yield controlled approaches to intersections	Tried
C3	Change horizontal/vertical alignment of approaches to provide more sight distance	Tried
D1	Provide an automated real-time system to inform drivers of the suitability of available gaps for making turning and crossing maneuvers	Experimental
D2	Provide roadside markers or pavement markings to assist drivers in judging the suitability of available gaps for making turning and crossing maneuvers	Experimental
E1	Improve visibility of intersections by providing enhanced signing and delineation	Tried
E2	Improve visibility of intersections by providing lighting	Proven
E3	Install splitter islands on the minor road approach to an intersection	Tried
E4	Provide a stop bar (or a wider one) on minor road approaches	Tried
E5	Install larger regulatory and warning signs at intersections	Tried
E6	Install rumble strips on intersection approaches	Tried
E8	Provide supplementary stop signs mounted over the roadway	Tried
E9	Provide pavement markings with supplementary messages (e.g., STOP AHEAD)	Tried
E10	Provide improved maintenance of stop signs	Tried
E11	Install flashing beacons at stop-controlled intersections	Tried
F1	Avoid signalizing through roads	Tried
F3	Provide roundabouts at appropriate locations	Proven
G1	Provide targeted enforcement to reduce stop sign violations	Tried
G2	Target public information & education on safety problems at specific intersections	Tried
H1	Provide targeted speed enforcement	Proven
H2	Provide traffic calming on intersection approaches through a combination of geometrics and traffic control devices	Proven
H3	Post appropriate speed limit on intersection approaches	Tried
I1	Provide turn path markings	Tried
I3	Provide lane assignment signing or marking at complex intersections	Tried

Note: Highlighted treatments were examined as case studies within Chapter 3 of this thesis.

Therefore, the objective of this project is to review and document expressway intersection safety countermeasures implemented by various STAs and evaluate their safety effectiveness, where possible, using a naïve before-after analysis methodology in an attempt to begin to understand their potential as rural expressway intersection safety treatments. This research will hopefully begin to set up a more proactive planning process for expressway intersection safety during expressway corridor development as well as a more comprehensive intersection countermeasure matrix. At low volumes, ordinary TWSC expressway intersections can provide very good safety performance and may be the most appropriate design in most locations when a two-lane roadway is first converted into an expressway. However, as volumes are projected to grow and site conditions are expected to change, effective intersection designs should be programmed well in advance of any safety and/or operational problems. This is the approach MoDOT is developing as shown in Table 5.

TABLE 5. MoDOT Expressway Intersection Planning Matrix (*6*)

	TYPE 1	TYPE 2	TYPE 3	TYPE 4	TYPE 5	TYPE 6
	No Turn Lanes	**With Turn Lanes**	**Offset Left-Turn Lanes**	**Median U-Turns**	**Partial Grade Separated Intersection**	**Interchange**
Mainline Volume	Conflicts can create rear-end crashes	Not a primary factor		Not a factor in selection, but a factor in design	Not a primary factor	
Crossroad Volume	< 10	< 2,000	< 3,000	< 4,000	> 3,000 *	> 4,000
Indirect Turning Movements	None			Some	None	
Recommended Median Width	60 feet				80 feet	N/A
ROW Impacts	Low			Medium	High	
Driver Expectation *1 (unmet) → 5 (met)*	2	5	4	2 or 3	3	5
Public Acceptance *1 (low) → 5 (high)*	3	5	4	2 or 3	4	5
Safety *1 (low) → 5 (high)*	1	2	3	4	4	5
Cost ** (*$1,000*)	$40 – $50	$100 – $150	$150 – $200	$100 – $250	$2500 – $3500	$5000 – $8000

* Insufficient data to determine upper threshold for Type 5 volumes.
** Cost assumption based on 60 foot median width on new construction. Does not include right-turn lanes or ROW acquisition.

1.4 RESEARCH APPROACH & THESIS ORGANIZATION

The approach to this research was divided into three major tasks. The first task was to perform a literature review to identify safe and effective median intersection design treatments. In order to identify the intermediate intersection design treatments between an ordinary TWSC rural expressway intersection (Figure 1) and an interchange, two important research questions must be addressed. First, the safety effectiveness or the expected safety improvement resulting from the implementation of each expressway intersection strategy must be quantified. Second, how traffic volume levels impact the safety and operational performance of each design strategy must be understood to determine the volume thresholds which trigger the planning and implementation of the next level of intersection improvement. Once more is known about the safety benefits of the various expressway intersection strategies and the impact of volume levels, a more proactive and systematic approach to expressway intersection safety planning can be developed. Therefore, the goal of the literature review was to determine what research has previously been conducted in these areas. A summary of these literature review findings is presented in Chapter 2.

Ten of the most promising expressway intersection safety strategies identified in the literature review were selected for further study. The second task in the research effort involved conducting "case studies" to investigate and document the experience of STAs who have experimented with these strategies. The case studies are presented in Chapter 3 and their objective was two-fold. The first objective was to interview knowledgeable staff from the respective agencies to determine: 1) the circumstances surrounding the treatment's implementation (i.e., reasons for, cost, etc.), 2) intersection site conditions (i.e., type and intensity of land use, traffic volumes and patterns, geometry including horizontal/vertical curves and skew, etc.), 3) public reaction (complaints, elderly driver issues, indications of erratic driving behavior, etc.), 4) issues the agency encountered, 5) lessons learned including design guidance and general advice, and 6) if the agency had performed any subjective or objective evaluations of the operational and/or safety performance of the treatment. The second objective of the case studies was to obtain crash data from the agency, where possible, and conduct naïve before-after analyses of the intersection safety treatments in order to begin to understand their potential for improving expressway intersection safety. By no means are these before-after analyses meant to be scientifically rigorous evaluations which develop reliable estimates of their safety effectiveness. They are simply observational before-after studies which compare the count of the before period crashes with the after period crashes.

This naïve before-after analysis methodology has two major limitations (*22*). First, it does not take regression-to-the-mean into account. The sites selected for treatment were not randomly selected. They were likely selected because they were high crash locations. Therefore, it is possible that, in the after period, there would have been a reduction in crashes even if nothing had been done due to simple chance. Only a Bayesian analysis can correct for the regression-to-the-mean bias (*22*). However, a secondary way to attempt to account for regression-to-the-mean is to use as many years of before and after crash data as possible, which is the approach taken in this research. Although some of the sites examined in the case studies had limited data available, where more than three years of before and after data were obtained, statistical evaluations were performed. The second limitation of the naïve before-after analysis approach is that the noted change in safety does not represent the true effect of the treatment. It also reflects the effect of other external factors such as changes in traffic volume, vehicle fleet mix, weather, driver behavior, and so on. It is not known what part of the change in safety can be attributed to the treatment and what part is due to changes in the various other external influences (*22*).

The third and final task of this research effort was to identify and describe expressway intersection safety topics warranting further study. A focus group was held in December of 2006 to prioritize future research interests regarding the ten countermeasures examined in the case studies. The results of this focus group prioritization as well as the conclusions and recommendations of this research effort are presented in Chapter 4.

CHAPTER 2: RURAL EXPRESSWAY INTERSECTION SAFETY LITERATURE REVIEW

2.1 OVERVIEW

This chapter provides a brief literature review of intersection safety treatments which have been applied at rural expressway/divided highway intersections. Two major questions were attempted to be answered through this review:

1) What intersection features have been found to influence the frequency and/or severity of rural expressway/divided highway intersection crashes? Of particular interest were studies analyzing intersection crash frequency, rate, or severity as a function of traffic demand, traffic control, or geometric design variables.

2) What intersection safety and/or operational corrective measures have been implemented at rural expressway/divided highway intersections, and, for each treatment found, is there any indication of its effectiveness in terms of reducing crash frequency and/or severity? How do traffic volume levels impact the safety and/or operational performance of each countermeasure?

The findings of previous research regarding these areas of interest are summarized in this chapter. Although most of the information presented herein is derived from prior research, the experience of various STAs is discussed in some cases to augment what is available in the existing literature.

2.2 SAFETY EFFECTS OF EXPRESSWAY INTERSECTION FEATURES

To improve the safety of at-grade intersections on rural expressways, the major factors contributing to the frequency and/or severity of crashes at these locations must be identified and understood; therefore, this became the focus for the first phase of the literature review. A number of the findings from this phase were previously discussed in the "Problem Statement" Section of Chapter 1. Table 6 provides a general summary of these findings and indicates the nature of the relationship found to exist between expressway/divided highway intersection safety and the particular traffic demand, traffic control, or geometric design element(s) investigated in each study. Within Table 6, the intersection features are divided into two major categories: geometric design elements and traffic control/operational elements. Each of these categories is then subdivided into elements which are related to the

TABLE 6. Expressway/Divided Highway Intersection Safety Literature Summary

Intersection Geometric Design Elements	Positive Correlation *	Negative Correlation	No Significant Effect
Number of Approach Legs	Harwood et al., 1995 (*9*)		
Intersection Skew/Angle	Van Maren, 1980 (*20*) Burchett & Maze, 2006 (*14*)		
Median Opening Length	Harwood et al., 1995 (*9*)		Van Maren, 1980 (*20*)
Total Distance to Cross Expressway	Van Maren, 1980 (*20*)		
Approach Geometric Design Elements	**Positive Correlation ***	**Negative Correlation**	**No Significant Effect**
Median Width (Expressway)		Priest, 1964 (*11*) Harwood et al., 1995 (*9*) [4-legged intersections] Maze et al., 2004 (*2*)	Cribbins et al., 1967 (*18*) Van Maren, 1980 (*20*) Harwood et al., 1995 (*9*) [3-legged intersections] Khattak et al., 2006 (*23*)
Presence of Median Barrier (Expressway)			Van Maren, 1980 (*20*)
Presence of Horizontal Curvature (Expressway)	Van Maren, 1980 (*20*) Burchett & Maze, 2006 (*14*) Khattak et al., 2006 (*23*)		
Presence of Vertical Curvature (Expressway)	Burchett & Maze, 2006 (*14*) Khattak et al., 2006 (*23*)		
Percent Grade (Expressway)			Van Maren, 1980 (*20*)
Number of Lanes (Expressway)	Harwood et al., 1995 (*9*) [4-legged intersections]	Harwood et al., 1995 (*9*) [3-legged intersections]	
Lane Width (Expressway)	Harwood et al., 1995 (*9*) [4-legged intersections]		Harwood et al., 1995 (*9*) [3-legged intersections] Khattak et al., 2006 (*23*)
Shoulder Width (Expressway)			Van Maren, 1980 (*20*) Harwood et al., 1995 (*9*)
Left-Turn Deceleration Lane Presence (Expressway)	Van Maren, 1980 (*20*) Harwood et al., 1995 (*9*) [3-legged intersections]	Harwood et al., 1995 (*9*) [4-legged intersections]	Maze et al., 2004 (*2*)
Left-Turn Median Acceleration Lane (MAL) Presence (Expressway)		Van Maren, 1980 (*20*) Hanson, 2002 (*24*)	
Offset Left-Turn Lane Presence (Expressway)	Schurr et al., 2003 (*25*) [due to speed differential]	Schurr et al., 2003 (*25*) [due to sight distance] Khattak et al., 2006 (*23*)	
Right-Turn Deceleration Lane Presence (Expressway)	Van Maren, 1980 (*20*)		Maze et al., 2004 (*2*)
Right-Turn Acceleration Lane Presence (Expressway)	Van Maren, 1980 (*20*)		
Intersection Traffic Control and Operational Elements	**Positive Correlation ***	**Negative Correlation**	**No Significant Effect**
Signalization	Cribbins et al., 1967 (*18*) Cribbins & Walton, 1970 (*19*) Van Maren, 1980 (*20*) Souleyrette & Knox, 2005 (*21*) [matched-pair analysis]	Solomon, 1959 (*17*) Souleyrette & Knox, 2005 (*21*) [before-after analysis]	
TWSC Beacon Presence		Solomon, 1959 (*17*)	
Median Traffic Control Type (stop/yield)			Harwood et al., 1995 (*9*)
Presence of Lighting	Harwood et al., 1995 (*9*)		
Presence of Rolling/Mountainous Terrain		Harwood et al., 1995 (*9*)	
Total Entering Volume (ADT)	Priest, 1964 (*11*)		McDonald, 1953 (*1*)

TABLE 6. Expressway Intersection Safety Literature Summary (continued)

Approach Traffic Control and Operational Elements	Positive Correlation ∗	Negative Correlation	No Significant Effect
Expressway Volume (ADT)	McDonald, 1953 (*1*) Priest, 1964 (*11*) Cribbins et al., 1967 (*18*) Bonneson & McCoy, 1992 (*12*) Harwood et al., 1995 (*9*) Maze et al., 2004 (*2*) Khattak et al., 2006 (*23*)		
Minor Road Volume (ADT)	McDonald, 1953 (*1*) Priest, 1964 (*11*) Bonneson & McCoy, 1992 (*12*) Harwood et al., 1995 (*9*) Maze et al., 2004 (*2*) Khattak et al., 2006 (*23*)		
Speed Limit (Expressway)	Cribbins et al., 1967 (*18*)		Khattak et al., 2006 (*23*)
Design Speed (Expressway)			Harwood et al., 1995 (9)
Advance Warning Signage (Expressway)			Van Maren, 1980 (*20*) Pant & Huang, 1992 (*26*)
Advance Warning Signage (Minor Road)		Van Maren, 1980 (*20*)	
Stop Sign Size (Minor Road)		Van Maren, 1980 (*20*)	
Presence of Painted Stop-Bars (Expressway & Minor Rd.)		Van Maren, 1980 (*20*)	
Access Control (Expressway)	Harwood et al., 1995 (*9*) [3-legged intersections]		Harwood et al., 1995 (*9*) [4-legged intersections]
Functional Classification (Divided Highway)		Harwood et al., 1995 (*9*) [4-legged intersections]	Harwood et al., 1995 (*9*) [3-legged intersections]

∗ A positive correlation means that as the element of interest increases in value or is present, crashes and/or crash surrogates increase according to the specified reference, indicating a deterioration of intersection safety. A negative correlation means that, according to the specified reference, crashes and/or crash surrogates decrease (i.e., safety improves) in the presence of the element of interest or as the element of interest's value increases.

intersection as a whole and those which may vary by intersection approach. To be included in Table 6, the research studies reviewed had to be performed on a homogenous sample of expressway/divided highway intersections. Intersections in rural areas were of particular interest; therefore, the studies included in Table 6 were those whose study sample contained at least some rural intersections or in which the area type was not specified.

Based on this review, many expressway intersection and approach characteristics seem to have the potential to influence the safety performance of these intersections; however, more research is required to definitively quantify their effects. On the other hand, it is interesting to note that few studies have investigated how minor road approach characteristics influence expressway intersection safety.

2.3 RURAL EXPRESSWAY INTERSECTION SAFETY TREATMENTS REVIEW

The second phase of the literature review was to examine the current state-of-the-practice of countermeasures used by STAs to improve intersection safety on rural expressways. STAs have experimented with a wide variety of intersection safety treatments at problematic rural expressway intersections. Most of these countermeasures are reactive measures which involve modifying (removing or adding) the intersection or approach features listed in Table 6. Unfortunately, it has been observed that STAs typically implement multiple countermeasures simultaneously, thereby making evaluation of individual measures difficult, if not impossible. As a result, the safety effects of individual treatments are rarely scientifically evaluated by STAs. Since rigorous before-after analyses have not been conducted, much of what is known about each treatment's effectiveness is based only on engineering judgment and subjective assessments. Therefore, ten rural expressway intersection safety treatments were selected for further study and the results of these "case studies" are described in Chapter 3. The purpose of this current section is to briefly introduce a number of the other rural expressway intersection safety treatments discovered in the literature review which are not addressed in the case studies.

In general, rural expressway intersection safety treatments can be divided into three broad categories: conflict point management techniques, gap selection aids, and intersection recognition devices. Conflict point management techniques are those treatments which remove, reduce, relocate, or control the 42 conflict points illustrated in Figure 2 which occur at a traditional TWSC rural expressway intersection. These techniques include grade separation, J-turn intersections, offset T-intersections, jughandle intersections, and signalization just to name a few. Gap selection aids are those countermeasures which aid a driver in selecting a safe gap into or through the expressway traffic stream. These aids include sight distance enhancements such as offset turn lanes, acceleration lanes which physically make it easier to merge into the expressway traffic stream, median pavement markings which promote a two-stage gap selection process, or other devices which detect and communicate the presence of a safe gap. Finally, intersection recognition devices are treatments such as warning signs and intersection lighting which improve intersection conspicuity for either the minor road or expressway drivers. Providing greater intersection

TABLE 7. Potential Rural Expressway Intersection Safety Treatments

Category	Subcategory	Treatment
Conflict Point Management Techniques	*Removal/Reduction Through Access Control*	1) Conversion of Entire Expressway Corridor to Freeway
		2) Isolated Conversion to Grade Separation or Interchange
		3) Close Low Minor Road Volume Intersections & Use Frontage Roads to Direct Traffic to Major Intersections
		4) Close Median Crossovers (Right-In, Right-Out Access Only)
		5) Convert Four-Legged Intersection into T-Intersection or Initially Construct T-Intersections instead of Four-Legged Intersections ∗ Offset T-Intersections ∗ Use a "One-Quadrant Interchange" Design (if necessary)
	Replacement of High-Risk Conflict Points	1) J-Turn Intersections (Indirect Minor Road Crossing & Left-Turns)
		2) Offset T-Intersections (Indirect Minor Road Crossing)
		3) Jughandle Intersections (Indirect Left-Turns Off Expressway)
		4) Other Indirect Left-Turn Treatments (Michigan Lefts)
		5) Expressway Semi-Roundabout Intersection (ES-RI)
	Relocation or Control	1) Provide Left/Right-Turn Lanes or Increase Their Length
		2) Provide Free Right-Turn Ramps for Exiting Expressway Traffic
		3) Minimize Median Opening Length
		4) Signalization
Gap Selection Aids	*Vehicle Detection (Intersection Sight Distance Enhancements)*	1) Provide Clear Sight Triangles
		2) Modify Horizontal/Vertical Alignments on Intersection Approaches
		3) Realign Skewed Intersections to Reduce or Eliminate Skew
		4) Move Minor Road Stop Bar as Close to Expressway as Possible
		5) Provide Offset Right-Turn Lanes
		6) Provide Offset Left-Turn Lanes
	Judging Arrival Time	1) Intersection Decision Support (IDS) Technology or Other Dynamic Device to Communicate Availability & Size of Gaps
		2) Roadside Markers/Poles (Static Markers at a Fixed Distance)
	Merging/Crossing Aids - - - - - - - - - - - - - *(Promoting Two-Stage Gap Selection)*	1) Provide Left-Turn Median Acceleration Lanes (MALs)
		2) Provide Right-Turn Acceleration Lanes
		3) Expressway Speed Zoning/Enforcement Near Intersections
		4) Widen Median to Provide for Adequate Vehicle Storage
		5) Add Centerline, Yield/Stop Signs/Bars, and Other Signage ("Recheck Cross Traffic Before Proceeding" or "Look" signs) in the Median
		6) Extend Left Edge Lines of Expressway Across Median Opening
		7) Public Education Campaign Teaching Two-Stage Gap Selection
Intersection Recognition Devices	*Intersection Treatments*	1) Provide Overhead Control Beacon Reinforcing Two-Way Stop Control
		2) Provide Intersection Lighting
	All Approaches	1) Enhanced (Overhead/Larger/Flashing) Intersection Approach Signage
	Expressway Approaches	1) Provide Enhanced Freeway-Style Intersection Guide Signs
		2) Provide Dynamic "Watch For Entering Traffic When Flashing" Signs or Other Activated Advance Intersection Warning Systems
		3) Use a Variable Median Width (Wider in Intersection Vicinity)
		4) Change Median Type in Vicinity of Intersection
	Minor Road Approaches	1) Use "Stop-Ahead" Pavement Marking & In-Lane Rumble Strips
		2) Provide a Stop Bar (or a Wider One)
		3) Provide Divisional/Splitter Island at Mouth of Intersection
		4) Provide Signage/Marking for Prevention of Wrong-Way Entry

Note: Highlighted treatments were selected for further study and are examined in Chapter 3 Case Studies.

recognition reduces the likelihood that a minor road driver will run the stop sign and alerts the expressway driver to proceed through the intersection with caution. Table 7 provides a categorized listing of numerous rural expressway intersection safety treatments as compiled from the literature review, including two previous surveys of STAs (*2, 27*). Some treatments fall into multiple categories, but have been placed in the category deemed most applicable.

In general, selection of the most appropriate safety countermeasure should be determined based on the crash types which tend to occur at each location. However, based on the apparent underlying cause of crashes at TWSC rural expressway intersections described in Chapter 1 (i.e., gap selection on the far-side by crossing and left-turning minor road drivers), the conflict point management techniques which remove the high risk conflict points associated with those minor road maneuvers and the gap selection aids would seem to have the most potential to improve rural expressway intersection safety.

2.3.1 Conflict Point Management Techniques

Intersection conflict points represent the locations where vehicle paths cross, merge, or diverge as they move from one intersection leg to another. Assuming opposing left-turn paths do not overlap, a typical TWSC rural expressway intersection has 42 conflict points as shown in Figure 2. Intersection conflict point analysis is a well understood means of comparing the expected safety of alternative intersection designs which suggests that the more conflict points an intersection design has, the more dangerous it will be (*28*). This approach is generally useful, but is ultimately limited because it assumes the crash risk is equal at each conflict point, when in fact, the crash risk associated with each conflict point varies depending on the complexity and volumes of the movements involved. The conflict points with the greatest crash risk (i.e., those accounting for the largest proportion of crashes) at TWSC rural expressway intersections tend to be the far-side conflict points involving minor road left-turns and crossing maneuvers (i.e., conflict points 15, 16, 19, 21, 22, and 25 in Figure 2).

Conflict point management techniques are those treatments which attempt to improve intersection safety by reducing, relocating, or controlling the number and/or type of vehicular conflicts that can occur at an intersection. The key to the effectiveness of these treatments however, is in eliminating the high-risk conflict points. Therefore, the conflict point management treatments with the most potential to improve rural expressway intersection safety are those which eliminate the far-side conflict points associated with minor road left-turns and crossing maneuvers or replace them with conflict points of lower risk and/or severity. In doing so, conflict point management techniques can be expensive and, because some movements become restricted, they tend to be the most controversial. Three conflict point management techniques (J-turn intersections, offset T-intersections, and jughandle intersections) were selected for further study and are discussed in detail as case studies within Chapter 3. A few other conflict point management techniques for TWSC rural expressway intersections are briefly discussed over the remainder of this section.

2.3.1.1 Grade Separation

The greatest safety, efficiency, and capacity of rural expressway intersections are attained when the intersecting roadways are grade separated because the conflict points are completely removed or drastically reduced if access is provided via an interchange. Interchanges provide the safest access to the expressway via high-speed ramps which create just a few merging/diverging conflict points on the mainline as opposed to the numerous direct entry crossing conflict points associated with at-grade intersections (*27*). However, the high cost of constructing a grade separation or an interchange limits their use on expressways to those locations where the additional expenditure can be justified. Chapter 10 of the Green Book (*3*) presents general warrants for converting at-grade intersections to grade separations or interchanges; however, no specific traffic volume or safety thresholds for conversion are provided. Bonneson and McCoy (*12*) developed more specific volume warrants for converting a TWSC expressway intersection into a full diamond interchange based on a benefit-cost analysis. The results indicated that a diamond interchange should begin to be considered once minor road volumes reach 2,000 vpd and is generally warranted by the time minor roadway volumes exceed 4,000 vpd.

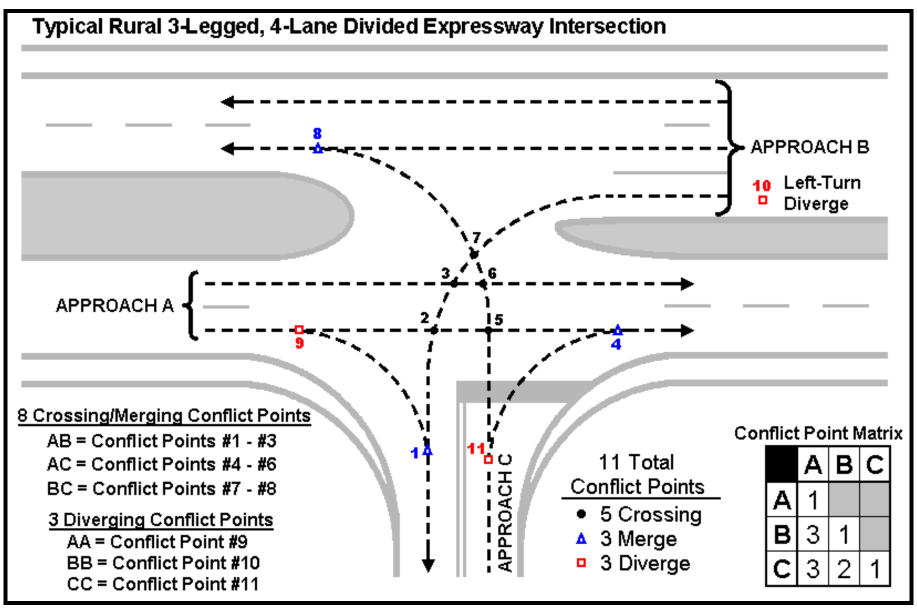
The majority of STAs consider interchanges to be a corrective measure for at-grade intersections with high crash rates and convert these intersections to interchanges on a case-by-case basis (*2, 27*). However, the mix of at-grade intersections and interchanges this practice creates along an expressway corridor may violate driver expectations; thus, according to a 2004 survey of 28 STAs (*2*), eight states had either constructed rural expressways that bypass cities as full access-controlled facilities (freeways) and/or upgraded expressways to freeways on a corridor basis. This practice involves converting all major intersections to interchanges and closing all other at-grade intersections as part of a contiguous project. Potential benefits of this strategy include the up-front dedication of right-of-way, provision of surplus capacity for future traffic growth, preservation of corridor access control, and design consistency. Of course, in the long run, this practice will be extremely expensive and impractical if widely applied. In the 2004 survey (*2*), the California, Texas, and North Carolina DOTs indicated that evaluation of current and projected route volumes (including minor roadways), level of service, and crash history are the major criteria they use to determine if conversion to full access control is appropriate along a particular expressway corridor.

2.3.1.2 Frontage Roads

In 1953, McDonald (*1*) developed the expressway intersection SPF given in Table 3. Based on his model, McDonald pointed out that when minor road volumes are less than approximately 2,400 vpd, the average crash frequency per minor road vehicle (crash risk) is reduced as the minor road volume increases. This observation led to the conclusion that the concentration of minor road traffic via the closing of low-volume crossroads and the provision of frontage roads may be an effective means of improving rural expressway intersection safety. To illustrate this concept, assume an expressway corridor carries 11,000 vpd and intersects six low volume county roads, each serving 100 vpd. Based on the McDonald SPF, each of these six intersections would be expected to have one crash per year, leading to a total of six intersection crashes per year along this particular expressway segment. On the other hand, if frontage roads are used to connect each of these county roads to a single expressway intersection serving the same 600 vpd, only three crashes would be expected to occur per year, thereby improving intersection safety along the corridor by fifty percent. More dramatic safety improvement occurs if the Maze et al. (*2*) SPF shown in Table 3 is used to apply this same example. These results make sense in terms of conflict point management because fewer intersections lead to fewer overall conflict points. Section 104.3 of the California Department of Transportation (CALTRANS) Highway Design Manual (*29*) states that an expressway frontage road is justified if the costs of constructing the frontage road are less than the costs of providing access via another means and should be provided when the number of access openings on one side of the expressway exceed three over a span of 1600 feet. Section 205.1 goes on to say that access openings on expressways should not be spaced closer than one-half mile of an adjacent public road intersection or another private access opening that is wider than 30 feet.

2.3.1.3 T-Intersections

It has long been acknowledged that three-legged intersections operate more safely than comparable four-legged intersections. Crash models developed by Harwood et al. (*9*) in 1995 revealed that crash frequency and rates at rural, three-legged, unsignalized, divided highway intersections are substantially lower than at their four-legged counterparts. Three-legged intersections (T-intersections) are less complex, lead to less driver confusion, and have almost 75 percent fewer conflict points at which conflicting traffic streams cross, merge, or diverge. A typical three-legged expressway intersection has only 11 total conflict points (see Figure 11) as compared with 42 at a typical four-legged expressway intersection (see Figure 2). However, more importantly, three-legged intersections eliminate the high-risk, far-side conflict points associated with crossing maneuvers made by minor road traffic and eliminate all but one of the far-side conflict points associated with minor road left-turns. In addition, the number of conflict points within the median crossover is dramatically reduced. Therefore, converting four-legged intersections into three-legged intersections or initially designing three-legged intersections instead of four-legged intersections during expressway corridor development should improve rural expressway intersection safety.



	A	B	C
A	1		
B	3	1	
C	3	2	1

FIGURE 11. Conflict point diagram for three-legged divided highway intersection

Where it is not reasonable or possible to eliminate the through movement on the minor road, two other possible design options exist which incorporate the use of T-intersections: an offset T-intersection or a one-quadrant interchange. Offset T-intersections are closely examined in the "Offset T-Intersection Case Study" presented in Chapter 3, while one-quadrant interchanges are described next.

2.3.1.4 One-Quadrant Interchanges

A one-quadrant interchange combines a T-intersection on the expressway with a grade separation to accommodate through traffic on the minor road as shown in Figure 12. The T-intersection is located at the terminal of a two-way ramp which serves all turning traffic. This design option thereby eliminates the right-angle crossing conflict points associated with through traffic on the two roadways, and only left-turning traffic travels through the median crossover at the T-intersection. Since only turning movements travel through the T-intersection, the traffic volume level through the at-grade intersection is typically much lower than what it would have been with a conventional TWSC intersection design. Lower volumes and fewer conflict points generally result in fewer crashes; thus the one-quadrant interchange is expected to have superior safety performance compared to a conventional TWSC expressway intersection. However, the Green Book (*3*) cautions that, with ramps in one quadrant, a high degree of channelization is normally needed at the ramp terminals, in the median, and on the through facility to properly and safely direct the turning maneuvers.

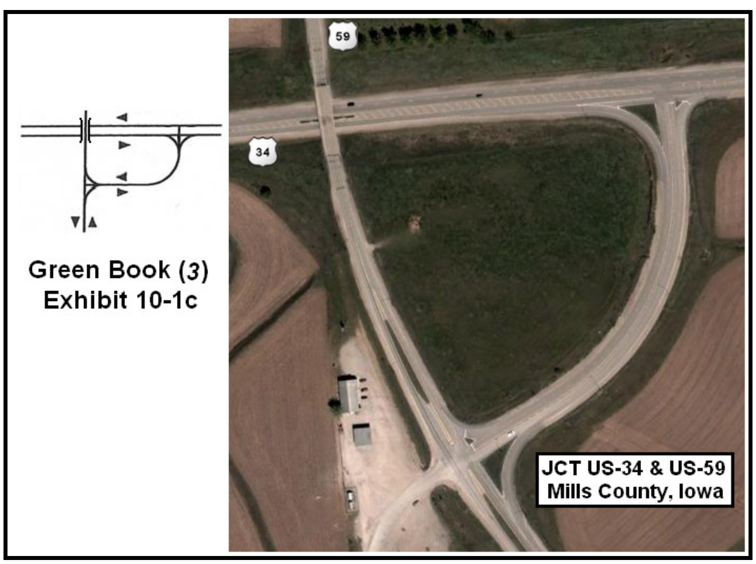


FIGURE 12. One-quadrant interchange

The IA-DOT has constructed two one-quadrant interchanges, one of which is shown in Figure 12. In both cases, the one-quadrant interchanges were originally planned as staged improvements prior to building a full interchange; however, the full interchanges were never constructed because additional safety and operational problems did not occur. Maze et al. (*2*) compared the actual crash severity indices of these one-quadrant interchanges with the crash severity indices that would have been expected had they remained TWSC intersections and found that the actual crash severity indices for the one-quadrant interchanges were about 60 percent less than expected for a conventional TWSC intersection with the same volumes. The approximate mainline volumes were 2,250 vpd and 3,270 vpd at the two locations with minor road volumes of approximately 1,300 vpd at both intersections.

The Green Book (*3*) states that appropriate candidate locations for a one-quadrant interchange are limited, but include intersections of roadways with low traffic volumes, intersections where ramp development is limited to one-quadrant because of topography or other constraints, and/or where it is constructed as the first step in the ultimate development of a full interchange. Other appropriate applications not listed in the Green Book may include: 1) locations where the minor road crossing volume exceeds what is considered acceptable for traditional at-grade intersection design strategies, or 2) locations where an interchange is not planned and is unlikely to be warranted due to traffic volumes, but problematic geometric conditions (horizontal/vertical curvature) and/or traffic patterns (hourly peaking) exist which have or are likely to result in a hazardous TWSC at-grade intersection. The major disadvantage of a one-quadrant interchange includes the cost of constructing the grade separation; therefore, it should only be considered once less costly alternatives have been examined.

2.3.1.5 Median U-Turn Intersections (Michigan Lefts)

The median U-turn intersection is illustrated in Green Book Exhibit 9-91 (Figure 13). Having typically applied this design at urban and suburban signalized intersections, the Michigan Department of Transportation (MDOT) is the most prominent user of this design in the U.S.; therefore, it is sometimes referred to as a “Michigan Left” or a “Michigan U-Turn” (*30*). Although not restricted through geometry, all direct left-turn maneuvers (from both the major and minor approaches) are prohibited via signage. All left-turn movements are thus made indirectly using the median U-turn crossovers (located immediately downstream from the main intersection) as indicated by the directional arrows in Figure 13. As a result, all left-turns at a median U-turn intersection must pass through the intersection twice. Since the indirect left-turns increase the volume of right-turns, exclusive right-turn lanes should be provided on all intersection approaches. In addition, the minimum median width depends on the selected design vehicle's U-turn radii requirements as indicated in Green Book Exhibit 9-92 (Figure 14). Finally, for signalized applications, the Green Book (*3*) states that the spacing from the main intersection to the U-turn location should be between 400 and 600 feet, while MDOT design standards propose 660 foot spacing (*30*). Ultimately, the selection of the most appropriate distance is a trade-off between providing sufficient U-turn storage (to minimize spillback potential) and safe/functional weaving areas while minimizing the travel distance/time of the indirect left-turn maneuvers (*30*).

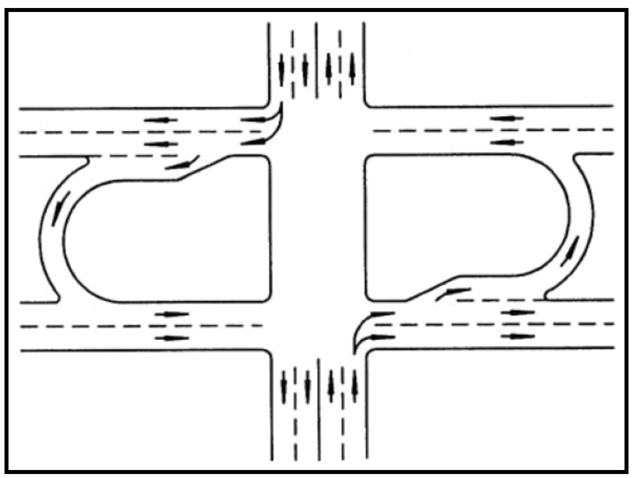
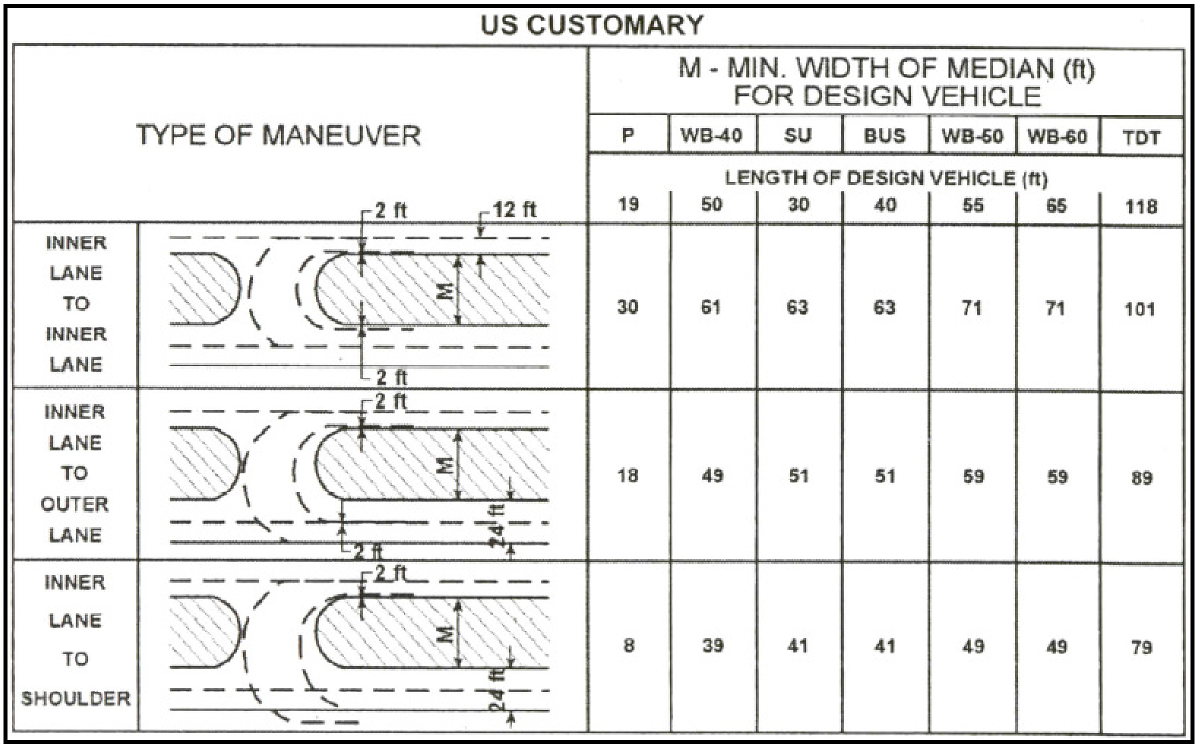


FIGURE 13. Green Book Exhibit 9-91; median U-turn intersection (3)

The original intent of the median U-turn intersection design was to increase capacity at high-volume signalized intersections. It does this by eliminating direct left-turn movements, thereby allowing two-phase signal operation. Although this design involves more "out-of-the-way" travel, it often reduces overall traffic delay and leads to level-of-service (LOS) improvements as compared to conventional signalized intersections (*30*). Furthermore, the intersection conflict points are reduced and more spread out. Studies conducted by MDOT have shown significant reductions in crashes (particularly right-angle crashes) as compared to conventional signalized intersections (*30*). However, their safety effectiveness in an unsignalized application at TWSC rural expressway intersections is still unknown. Some potential disadvantages of the median U-turn intersection design include driver confusion, driver disregard of the left-turn prohibitions, increased travel distance for left-turning traffic, and additional ROW requirements.



US CUSTOMARY							
TYPE OF MANEUVER	M - MIN. WIDTH OF MEDIAN (ft) FOR DESIGN VEHICLE						
	P	WB-40	SU	BUS	WB-50	WB-60	TDT
	LENGTH OF DESIGN VEHICLE (ft)						
	19	50	30	40	55	65	118
INNER LANE TO INNER LANE	30	61	63	63	71	71	101
INNER LANE TO OUTER LANE	18	49	51	51	59	59	89
INNER LANE TO SHOULDER	8	39	41	41	49	49	79

FIGURE 14. Green Book Exhibit 9-92; minimum designs for U-turns (3)

The median U-turn intersection has many similarities with the J-turn (superstreet) intersection detailed in the "J-Turn Intersection Case Study" presented in Chapter 3; however, they have some important differences and the two designs should not be confused. The major difference is that the median U-turn intersection allows minor road through traffic to travel straight through the median while the J-turn intersection utilizes a directional median opening which forces all minor road traffic to turn right. In addition, the median U-turn intersection requires indirect left-turn exits from the divided highway, while the directional median opening at a J-turn intersection allows direct left-turn exits at the main intersection.

2.3.1.6 Signalization

Signalization of rural expressway intersections was briefly discussed in the "Research Objectives" section of Chapter 1. Traffic signals are classified as a conflict point management strategy because they attempt to control intersection conflict points by alternately assigning the right-of-way between conflicting movements; however, they clearly serve as a gap selection aid and as an intersection recognition device as well.

In most instances, traffic demand from the minor road is low to moderate and intersections on rural expressways are adequately served by TWSC. However, as population and development increase, the control of access along an expressway will eventually concentrate traffic demands at intersections to the extent that elevated crash potential and/or excessive delay are experienced by minor road vehicles. When this begins to occur, public outcry and political pressure for signalization is inevitable. However, contrary to public perception, traffic signals are not a cure-all solution for intersection safety, especially on rural expressways. Comparing the crash frequencies predicted by SPFs for signalized versus TWSC expressway intersections developed by Bonneson and McCoy (*12*) reveals that, when expressway volumes are between 7,000 and 15,000 vpd with minor road volumes ranging from 100 to 4,000 vpd, signalized intersections are expected to have more crashes until the minor road volume level reaches approximately one-fourth of the expressway volume. However, when this occurs, Bonneson and McCoy concluded that a diamond interchange is a more economically viable alternative than signalization. Moreover, large variability in the safety effectiveness of signalization at individual rural expressway intersections in Iowa has been observed (*21*) and current research does not provide a decisive conclusion as to whether or not signalization will improve safety at rural expressway intersections.

Traffic signals can be dangerously inconsistent with the expressway driver's expectation of a free flow roadway for high-speed travel, thus creating increased potential for rear-end crashes and red-light running. As such, the AASHTO Green Book (*3*) and NCHRP 500, Volume 5 (*16*) guard against the use of signalization as a safety device and state that intersection control by traffic signals should be avoided. Therefore, according to a 1993 survey of STAs (*27*), some states have established policies that prohibit signalizing expressway intersections altogether, while other states do so only after other alternatives have been considered first. Other criteria mentioned in the decision to use traffic signal control at rural expressway intersections included crash rates, traffic volume levels, and median width.

2.3.2 Gap Selection Aids

As described in Chapter 1, right-angle collisions are the primary safety issue at TWSC rural expressway intersections. The predominant causes for these crashes seem to be the failure of minor road drivers to detect approaching expressway traffic or their inability to adequately judge the speed and distance (i.e., arrival time) of oncoming expressway vehicles. These gap selection issues may be exacerbated by the presence of certain intersection geometric features (i.e., horizontal/vertical curvature on the mainline, intersection skew, median width, etc.), driver age, driver behavior (i.e., one-stage gap selection), and increasing traffic volumes on both of the intersecting roadways.

Gap selection aids are those countermeasures which are intended to aid a driver in selecting a safe gap into or through the expressway traffic steam. Gap selection is a complex process which first involves vehicle detection. If an oncoming vehicle is spotted, the driver must then assess the size of the gap (i.e., time-to-arrival of the approaching vehicle) and determine whether or not there is enough time/space to complete their desired maneuver. If there is, the driver must proceed and physically enter or cross through the expressway traffic stream. Therefore, gap selection aids can generally be classified into three groups; those countermeasures which are intended to aid a driver with: 1) vehicle detection (i.e., intersection sight distance (ISD) enhancements), 2) judging the arrival time of oncoming vehicles, and 3) physically merging into or crossing through the expressway traffic stream. The concept of gap selection being a key contributing factor in rural expressway intersection collisions appears to be a recent idea. As a result, there are relatively few countermeasures to assist drivers with judging the arrival time of oncoming vehicles. Overall, five gap selection aids (IDS technology, static roadside markers, MALs, offset right-turn lanes, and offset left-turn lanes) were selected for further study and are examined in detail as case studies within Chapter 3. Other gap selection aids for TWSC rural expressway intersections are briefly introduced over the remainder of this section.

2.3.2.1 Maximize Intersection Sight Distance (ISD)

A long recognized intersection safety and operations principle is that the driver of a vehicle approaching an intersection should have an unobstructed view of the entire intersection, including all traffic control devices, and if stopped, sufficient unobstructed sight-lines should be provided along the intersecting highway to permit the driver to anticipate and avoid any potential collisions (*3*). The lack of adequate ISD at TWSC rural expressway intersections may hinder the ability of minor road drivers to detect oncoming expressway vehicles and/or adequately judge the suitability of available gaps in the expressway traffic stream when making turning or crossing maneuvers (*16*). Therefore, maximizing ISD is one possible strategy to aid minor road drivers with gap selection at TWSC rural expressway intersections, thereby minimizing the possibility that traffic, the road, or the roadside environment may distort, block, or distract motorist vision.

Clear departure sight triangles, triangular areas free of obstructions which may block a minor road driver's view of oncoming traffic as shown in Figure 15, should provide sufficient ISD for a stopped driver to make a decision to proceed, depart from the intersection, and complete the desired maneuver without collision (*3*). Conversely, they would allow drivers on the expressway to see any vehicles stopped on the minor road approaches so that the expressway driver will be prepared to slow or stop if necessary. The minimum recommended dimensions for the legs of clear departure sight triangles are described in Chapter 9 of the AASHTO Green Book (*3*) and are based on the type of traffic control used at the intersection, the type of maneuver to be performed, the design speed and grade of the intersecting roadway, and observations of driver gap acceptance behavior. If the available sight distance along the expressway (*b* in Figure 15) is at least equal to the stopping sight distance (SSD) for an expressway vehicle, then all drivers should have sufficient visibility to anticipate and avoid collisions; however minimum AASHTO ISD for stop-controlled intersections is longer than SSD to ensure the intersection operates smoothly (*3*). While estimates of the safety effectiveness of providing full ISD where it does not currently exist suggest that up to a twenty percent reduction in related crashes can be expected (*16*), no studies were found

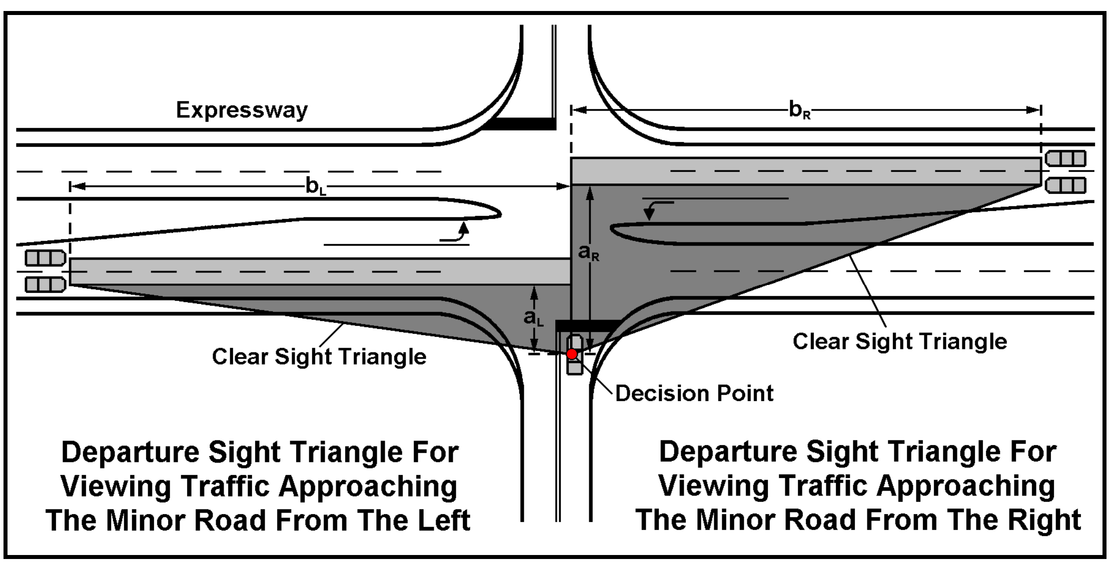


FIGURE 15. Clear departure sight triangles for TWSC expressway intersection

which examined the relationship between the amount of available ISD and the frequency and/or severity of collisions at TWSC rural expressway intersections.

A number of other intersection design features can play a major role in determining the amount of available ISD at an expressway intersection such as intersection skew, horizontal and vertical alignment, the type of left and right-turn lanes provided, as well as the location of the minor road stop bar. The Green Book (*3*) "driver gap-acceptance behavior method" for determining ISD includes some minor adjustments for intersection skew and vertical grades, but does not provide any adjustment factors for horizontal curvature. In addition, field observations of vehicle stopping positions found that minor road drivers will stop with the front of their vehicles placed 6.5 feet or less from the edge of the major road traveled way (*3*). As a result, the Green Book (*3*) method for determining ISD assumes 10 feet in order to provide a larger departure sight triangle. Section 3B.16 of the *Manual on Uniform Traffic Control Devices* (MUTCD) (*31*) gives the following guidance for the placement of stop bars:

> *In the absence of a marked crosswalk, the stop line should be placed at the desired stopping point, but should be placed no more than 30 feet nor less than 4 feet from the nearest edge of the intersecting traveled way. Stop lines should be placed to allow sufficient sight distance to all other approaches of an intersection.*

FIGURE 16. Expressway intersection where minor road stop bar could be moved forward

Figure 16 illustrates an expressway intersection in Nebraska where the minor road stop bar is placed too far back and could be moved forward to enhance ISD. In doing so, the base distance of the departure sight triangles (*a* in Figure 15) will be minimized and the available ISD will be maximized if the stop bars, in conjunction with stop signs, are able to inform minor road drivers of the proper location to stop. This strategy was first suggested by Van Maren (*20*) in 1980 after crash models in his research showed that crash rates for 39 randomly selected, multi-lane divided highway intersections in rural Indiana increased as the total distance across the divided highway increased. This strategy was reiterated by Agent (*32*) in 1988 after an investigation of the collisions and site characteristics at 65 high-speed intersections in rural Kentucky.

2.3.2.2 Right-Turn Acceleration Lanes (RTALs)

The minimum required departure sight triangle for a right-turn from a stopped approach onto a major uncontrolled roadway (Green Book ISD Case B2) typically requires more ISD than for a crossing maneuver from the same minor road approach (Green Book ISD Case B3). Therefore, if it is only feasible to provide the minimum required ISD for Case B3, if there is a large right-turn volume (especially trucks), if there are limited gaps in the expressway traffic stream, or if there is a high proportion of rear-end, sideswipe, or broadside collisions related to right-turn maneuvers from the minor road, a right-turn acceleration lane (RTAL) may be necessary. RTALs move the right-turn merge conflict point downstream and allow right-turning traffic to accelerate to expressway speeds before performing a high-speed merge to join the expressway traffic stream. This maneuver should thereby make gap selection easier/safer and reduce delay by replacing a low-speed, direct entry, right-angle turn. However, in a 1980 study, Van Maren (*20*) found that divided highway intersection crash rates tend to be higher where RTALs are present. No other studies were found on this issue and the safety effectiveness of RTALs at rural expressway intersections is still unknown (*16*). Figure 17 shows a RTAL in Minnesota which is designed with a larger turning radius, channelization, yield control, and a tapered-type entry. The top portion of Figure 17 shows the RTAL from the minor road approach, while the bottom portion shows the acceleration lane as it lies adjacent to the expressway.

The design guidance for acceleration lanes at intersections within Chapter 9 of the AASHTO Green Book (*3*) is limited. However, it does state:

> *Acceleration lanes are not always desirable at stop-controlled intersections where entering drivers can wait for an opportunity to*

merge without disrupting through traffic. Acceleration lanes are advantageous on roads without stop control and on all high-volume roads even with stop control where openings between vehicles in the peak-hour traffic streams are infrequent and short.

FIGURE 17. Right-turn acceleration lane in Minnesota

The Green Book (*3*) then refers the reader to Chapter 10, “Grade Separations and Interchanges”, for guidance related to acceleration lane lengths. Chapter 10 describes the differences between taper-type and parallel-type entrance terminals and gives the minimum acceleration lengths required at entrance terminals in Exhibit 10-70 (Figure 18) and adjustment factors for grades in Exhibit 10-71 (Figure 19). However, the Green Book goes on to state:

There should be additional length to permit adjustments in speeds of both through and entering vehicles so that the driver of the entering vehicle can position his or her vehicle opposite a gap in the through traffic stream and maneuver into it before reaching the end of the lane.

US Customary										
		Acceleration length, L (ft) for entrance curve design speed (mph)								
Highway		Stop condition	15	20	25	30	35	40	45	50
Design speed, V (mph)	Speed reached, V_a (mph)	and initial speed, V'_a (mph)								
		0	14	18	22	26	30	36	40	44
30	23	180	140	—	—	—	—	—	—	—
35	27	280	220	160	—	—	—	—	—	—
40	31	360	300	270	210	120	—	—	—	—
45	35	560	490	440	380	280	160	—	—	—
50	39	720	660	610	550	450	350	130	—	—
55	43	960	900	810	780	670	550	320	150	—
60	47	1200	1140	1100	1020	910	800	550	420	180
65	50	1410	1350	1310	1220	1120	1000	770	600	370
70	53	1620	1560	1520	1420	1350	1230	1000	820	580
75	55	1790	1730	1630	1580	1510	1420	1160	1040	780

Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 ft.

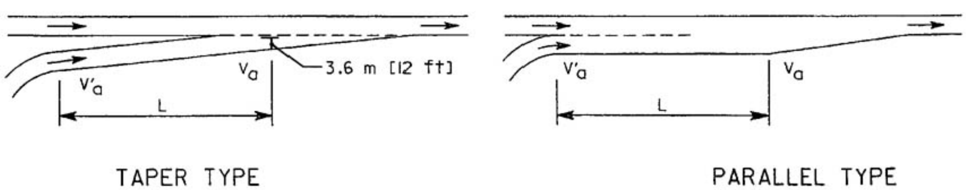


FIGURE 18. Green Book Exhibit 10-70; minimum acceleration lengths for entrance terminals with flat grades of two percent or less (3)

US Customary					
Design speed of highway (mph)	Acceleration lanes Ratio of length on grade to length of level for design speed of turning curve (mph)[a]				
	20	30	40	50	All speeds
		3 to 4% upgrade			3 to 4% downgrade
40	1.3	1.3	—	—	0.7
45	1.3	1.35	—	—	0.675
50	1.3	1.4	1.4	—	0.65
55	1.35	1.45	1.45	—	0.625
60	1.4	1.5	1.5	1.6	0.6
65	1.45	1.55	1.6	1.7	0.6
70	1.5	1.6	1.7	1.8	0.6
		5 to 6% upgrade			5 to 6% downgrade
40	1.5	1.5	—	—	0.6
45	1.5	1.6	—	—	0.575
50	1.5	1.7	1.9	—	0.55
55	1.6	1.8	2.05	—	0.525
60	1.7	1.9	2.2	2.5	0.5
65	1.85	2.05	2.4	2.75	0.5
70	2.0	2.2	2.6	3.0	0.5

[a] Ratio from this table multiplied by the length in Exhibit 10-70 or Exhibit 10-73 gives length of speed change lane on grade.

FIGURE 19. Green Book Exhibit 10-71; acceleration lane adjustment factors as a function of grade (3)

One potential concern with RTALs pointed out by NCHRP 500, Volume 5 (*16*) is that through expressway drivers may mistake them for an additional through lane if the RTAL is excessively long or poorly marked. MoDOT has addressed this concern by placing delineators between the RTAL and the through expressway lanes as shown in Figure 20.

FIGURE 20. Right-turn acceleration lane delineators used in Missouri

2.3.2.3 Expressway Speed Zoning

Another possible gap selection aid is to reduce the expressway speed limit through rural expressway intersections (i.e., speed zoning). A speed zone is defined as "A section of a street or highway where the speed limit is different from the statutory speed limit that has been established for the rest of the facility (*33*)." The purpose of speed zoning is to establish a speed limit which is "reasonable and safe for a given section of roadway (*33*)." This strategy is a gap selection aid because it increases the time-to-arrival (TTA) of an approaching expressway vehicle (i.e., it increases the available time gap for a minor road vehicle to enter or cross the expressway). The success of this strategy assumes, of course, that a direct relationship exists between the posted speed limit and the operating speed of expressway traffic, which may not be valid, especially for a short zone around an intersection (*33*).

Human factors research has attempted to study the motion perception ability of drivers in relation to their gap acceptance judgments and found that drivers, especially those 56 years of age and older, tend to rely more on their instantaneous judgment of the distance to oncoming vehicles rather than their estimated approach speeds or TTA when making gap selection decisions in left-turn leaving situations (i.e., longitudinal gap selection through head-on traffic) (*34*). No such research was found for gap selection while making turning or crossing maneuvers from the minor road (i.e., lateral gap selection through side-to-side traffic). However, let's assume for a moment that 1) this finding holds true for lateral gap selection, and 2) the critical time gap for a crossing maneuver from the minor road is hypothetically equal to 10 seconds. Now imagine two side-by-side approaching expressway vehicles are each 1000 feet away from an intersection where a minor road vehicle is stopped and waiting to cross. According to the first assumption, the minor road driver is going to base his or her decision to proceed on the 1000 foot distance, regardless of the speed of the approaching vehicles. However, if one of the oncoming expressway vehicles is traveling at 55 mph and the other is approaching at 75 mph, their TTA will be 12.4 and 9.1 seconds, respectively. Therefore, if the minor road driver identifies 1000 feet as a safe gap distance and proceeds to cross the expressway, there will be a collision with the vehicle approaching at 75 mph according to the second assumption. Therefore, reducing the speed of oncoming expressway vehicles would prolong the gaps selected by minor road drivers, allowing them more time to maneuver into or through those gaps.

Research on the effectiveness of reduced speed limits (speed zoning) or speed advisory through rural expressway intersections in reducing the frequency and/or severity of collisions is scarce. Iowa saw a reduction in average expressway speeds during peak hours after an advisory speed limit ten mph below the posted speed was placed in advance of an expressway intersection; however, related intersection crash data were not examined (*2*). In a much older study, Cribbins et al. (*18*) found that higher posted speed limits did lead to increased crash frequency along 92 rural and urban divided highway segments.

2.3.2.4 Median Widening

The Green Book (*3*) defines median width as, "The dimension between the inside edges of traveled way which includes the left interior shoulders, if any." This definition thereby includes any median deceleration or acceleration lanes as part of the median width. On rural expressways, the median serves many functional purposes; however, its major objectives are different between intersections versus at intersections. Between intersections, the major function of the median is to separate opposing expressway traffic. According to the

Green Book (*3*), a median width of 40 feet or wider will allow expressway drivers to experience a sense of separation from opposing traffic (i.e., noise, air pressure, and headlight glare from opposing traffic are drastically reduced).

In contrast, the major function of the median at intersections is to provide a refuge area for left-turning expressway traffic as well as for left-turning and crossing traffic from the minor road. The Green Book (*3*), NCHRP 375 (*9*), and NCHRP Synthesis 281 (*35*) address key factors in selecting the appropriate median width at rural divided highway intersections and present many advantages and disadvantages related to both narrow and wide medians. However, research has shown that four-legged, TWSC rural expressway intersections with wider medians are safer (*2, 9, 11*). SPFs developed by Harwood et al. (*9*) and Maze et al. (*2*) estimated 1.22 and 0.74 percent reductions in annual crash frequency with every one foot increase in median width, respectively. This is most likely due to the fact that wider medians allow for two-stage gap selection (i.e., a minor road left-turning or crossing driver can safely stop in the median area to evaluate the adequacy of the gap in expressway traffic coming from the right, thereby reducing the relative crash risk associated with these maneuvers). As a result, the Green Book (*3*) recommends that medians at TWSC rural expressway intersections generally be "as wide as practical" and should, at a minimum, be wide enough to store the design vehicle selected for making left-turning and crossing maneuvers from the minor road. The Green Book goes on to state:

> *Where a median width of 25 feet or more is provided, a passenger car making a turning or crossing maneuver will have space to stop safely in the median area. Medians less than 25 feet should be avoided at rural intersections because drivers may be tempted to stop in the median with part of their vehicle left unprotected from through traffic. The school bus is often the largest vehicle to use the median roadway frequently. The selection of a school bus as the design vehicle results in a median width of 50 feet. Larger design vehicles, including trucks, may be used at intersections where enough turning or crossing trucks are present; median widths of 80 feet or more may be needed to accommodate large tractor-trailer trucks without encroaching on the through lanes of a major road.*

While the statement above is technically correct (i.e., a 25 foot median will provide enough room to fully store a 19 foot passenger car with three feet of clearance from the through expressway lanes), a passenger car stored in such a median would block the left-turn leaving paths of exiting expressway vehicles considering a four-legged intersection with traditional twelve foot wide left-turn lanes on each expressway approach as illustrated on the left-hand side of Figure 21. This may make a left-turning or crossing minor road driver more reluctant to stop in the median area, thereby increasing the probability of one-stage gap selection behavior. In order to promote the safer two-stage gap selection process for the driver of a 19 foot passenger car, the median width at such an intersection should be at least 37 feet as shown on the right-hand side of Figure 21. This dimension would create enough space to store the passenger car in the median with three feet of clearance from the expressway left-turn lane at its front and from the expressway through lane at its rear. However, it may be a good idea to provide additional median width to allow more of the passenger car's deceleration to take place within the median as it comes to a stop after crossing the near-side expressway lanes. Based on this discussion, the 50 foot minimum median width for a school bus specified in the Green Book should be re-examined as well. If offset left-turn lanes are provided on the expressway (see the "Offset Left-Turn Lanes Case Study" in Chapter 3), a different minimum median width may be required.

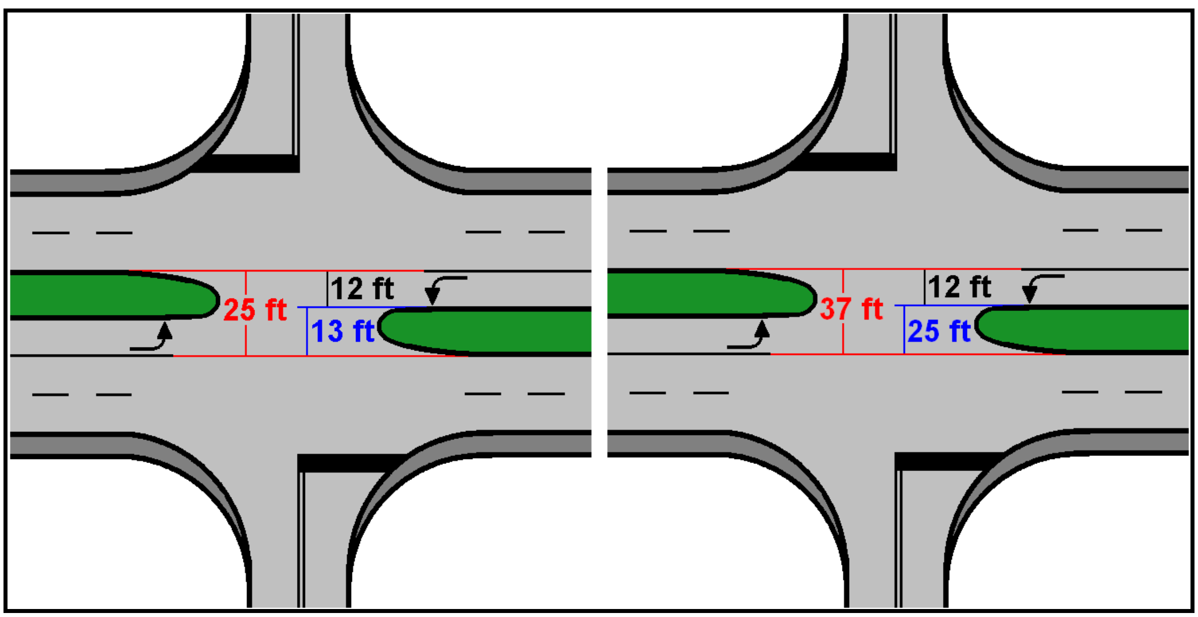


FIGURE 21. Expressway intersection median width/storage illustration

Another consideration in selecting the appropriate median width at rural expressway intersections is the turning behavior of opposing left-turn drivers. Field data examined in NCHRP 375 (*9*) suggested that opposing left-turn drivers leaving the expressway tend to turn in front of one another (i.e., simultaneous left-turns) when the median width is 50 feet or less, but tend to turn behind one another (i.e., interlocking left-turns) when the median width is greater than 50 feet. Figure 22 illustrates both types of left-turn behavior. There is no implication that one behavior is more desirable than the other; however, this finding may make 50 feet an appropriate breakpoint when setting design policies for selecting median widths at rural expressway intersections with traditional left-turn lanes (offset left-turn lanes dictate simultaneous left-turn behavior). The turning behavior of opposing left-turn leaving drivers in traditional left-turn lanes may also be affected by the median opening length (defined in Figure 23); however this relationship was not examined in NCHRP 375 (*9*); nor was the turn behavior of opposing minor road left-turn drivers.

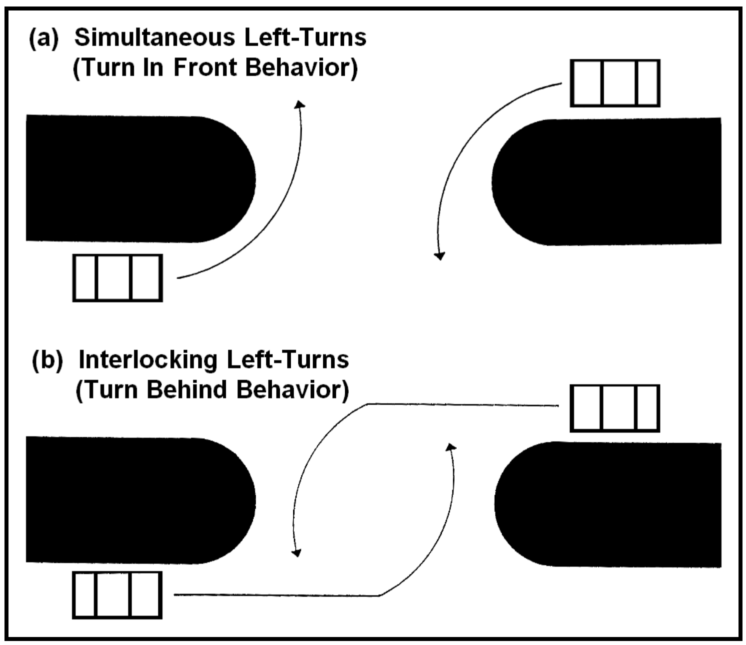


FIGURE 22. Opposing left-turn leaving driver behavior paths (9)

In 1995, NCHRP 375 (*9*) conducted a survey examining the design policies and practices of STAs related to median design and found that only 42 percent of the responding agencies had minimum median width standards of greater than 30 feet for rural non-freeway divided facilities. However, 76 percent of the responding agencies reported that they consider intersection operations when selecting the median width for a divided highway corridor, 50 percent indicated that storage needs in the median area influence their median width policy, and 62 percent indicated a desirable median width of more than 50 feet. In most states, rural expressway intersection median width is generally governed by the width of

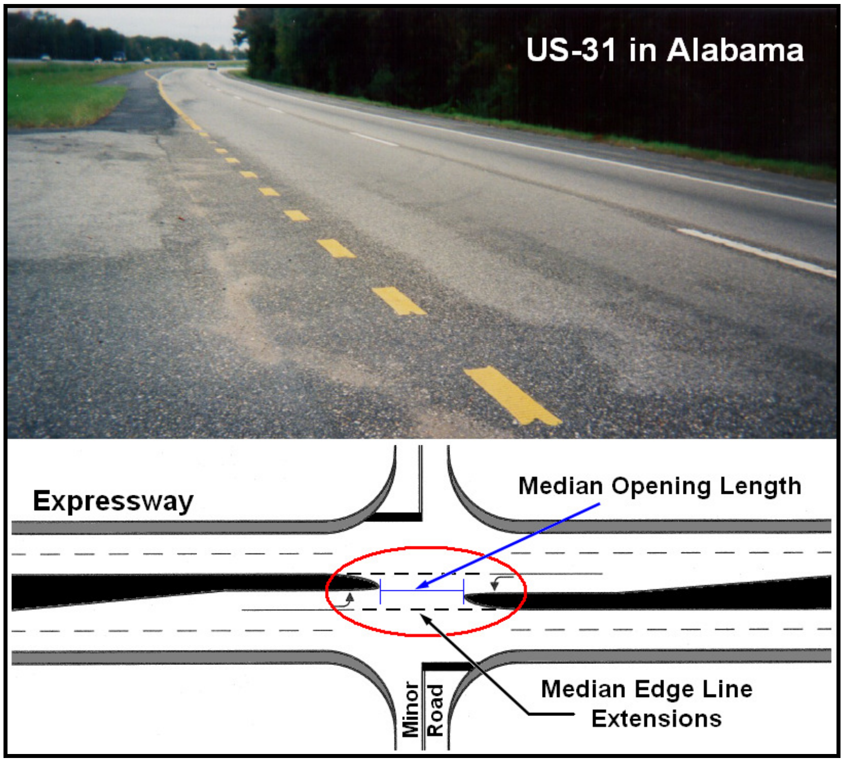


FIGURE 23. Edge line extensions through median crossover

median selected for the entire expressway corridor. However, Chapter 7, Section 5.7.5 of the Kansas Department of Transportation (KDOT) Design Manual (*36*) includes standards for rural expressway median widening (up to 150 feet from their standard median width of 60 feet) in the vicinity of intersections as shown in Figure 24. KDOT considers this type of treatment at divided highway intersections when the projected minor road volumes are in the 800 to 1000 vpd range with a high percentage of trucks, when mainline traffic operates at LOS C or worse, if ISD is extremely limited, or at other intersections with U.S. or Kansas State routes serving major traffic generators such as schools or industrial areas that cause a large amount of hourly peaking, but where interchanges have not been deemed necessary. Besides providing the benefit of extra median storage, wider medians in the vicinity of intersections could also serve as an intersection recognition device for expressway traffic by emphasizing the presence of the upcoming intersection.

The safety effectiveness of this treatment has not yet been evaluated; however, 20 STAs responding to the NCHRP 375 (*9*) survey reported operational problems at intersections related to medians that were considered to be "too wide", including the increased potential for wrong-way entry, especially at night. On the contrary, statistical

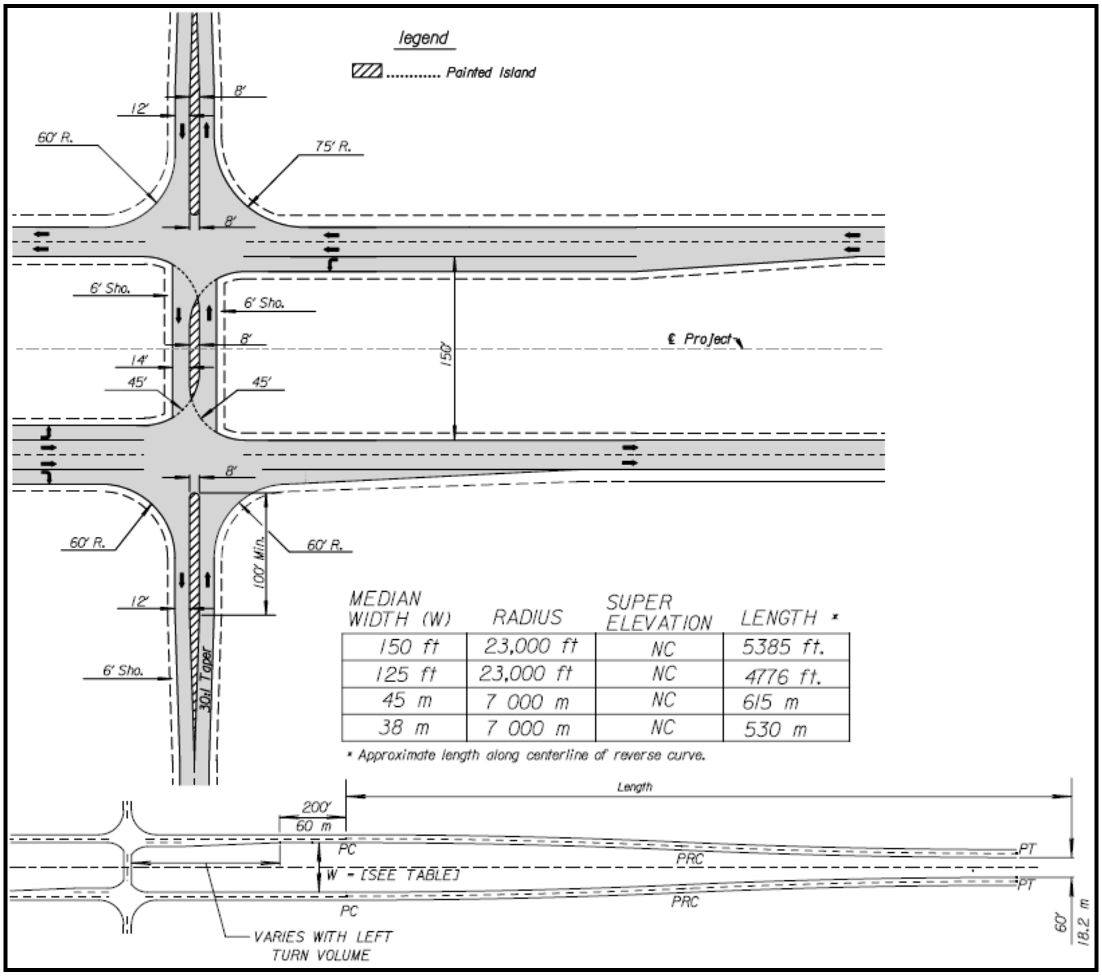


FIGURE 24. KDOT standard plan for wide median intersection (36)

analysis of the correlation between median width, median opening length, and the rate of undesirable maneuvers observed per 1,000 vehicles entering the median roadway during field observational studies conducted for NCHRP 375 at 20 unsignalized rural divided highway intersections in 8 different states revealed that the rate of undesirable maneuvers decreased as the median width increased. This suggests that as the median width of a divided highway becomes wider, fewer operational problems are observed at the intersections; however the median opening length should not be unnecessarily large as the same analysis revealed that the rate of undesirable maneuvers increased as the median opening length increased. In other words, the geometrics of a wide median in combination with a smaller median opening help create the impression that there is not much choice in traversing the median except to follow the path the designer intended (*9*). The desired path can also be emphasized through median delineation.

2.3.2.5 Median Signage & Delineation

Median signage and delineation have four major objectives: 1) to inform minor road drivers that they have reached a divided highway intersection, 2) to establish the right-of-way between median and far-side expressway traffic, 3) to communicate the appropriate gap selection process (i.e., one or two-stage), and 4) to define the proper travel paths through the median roadway. When the median is wide enough to store a passenger car, stop or yield bars in conjunction with stop or yield signs should be present in the median to establish right-of-way and to communicate the appropriate two-stage gap selection behavior to the minor road driver. Generally, median stop control is used when the selected design vehicle (usually a 40 foot school bus) can be completely stored within the median area. This marking and signing scheme, along with the use of a double yellow median centerline, is shown in MUTCD (*31*) Figure 2B-13 (Figure 25) where the median is 30 feet or wider, but not in MUTCD (*31*) Figure 2B-14 (Figure 26) where the median width is less than 30 feet. The reason behind the selection of this 30 foot median width threshold (42 feet by the Green Book median width definition) for deciding whether or not to use this median marking and signing scheme is not stated in the MUTCD and should be re-examined; however, the 30 foot value was likely selected based on the experience of STAs rather than on any particular research (*9*). Nevertheless, the signing/marking scheme shown in Figure 25 effectively provides a measure of depth perception to communicate to the minor road driver that the median is wide enough for vehicle storage, thereby promoting two-stage gap selection behavior. When the median width is not wide enough to store a passenger car, requiring one-stage gap selection by all vehicles, the signing/marking scheme shown in Figure 26 should be used.

Often, rural expressway intersections with wide medians have large expanses of pavement that can make it difficult for drivers to decide what path to follow and to anticipate the paths other drivers will take. The presence of a double yellow median centerline is also expected to help provide visual continuity with the centerline of the minor road approaches and help define the desired vehicle paths through the median roadway (i.e., the double yellow centerline reinforces turn behind behavior for opposing left-turns from both the expressway and minor road approaches), which in-turn, helps to reduce the number of undesirable driving behaviors (i.e., side-by-side queuing, angle-stopping, and through lane encroachment) and

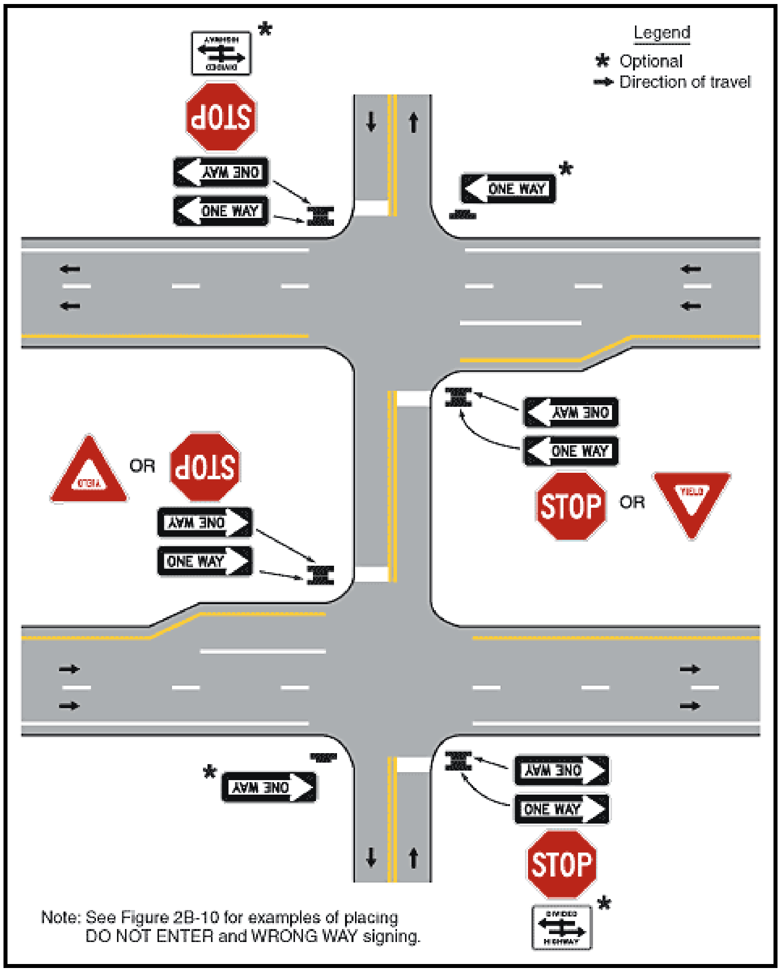


FIGURE 25. MUTCD Figure 2B-13; example of ONE WAY signing for divided highways with medians of 30 feet or greater (31)

conflicts occurring in the median area (*16*). The safety effectiveness of providing stop or yield bars, stop or yield signs, and/or a double yellow centerline within the median of rural expressway intersections has not been quantified. However, this strategy has been used in many low volume rural expressway intersection medians across Iowa. Limited before-after crash analysis has shown a reduction in intersection-related crashes following the introduction of this type of median signage and delineation (*2*). After the median pavement markings wore off, the crash rate tended to increase; therefore, the IA-DOT has proposed using milled-in tape median pavement markings at these locations in the future.

Other types of median signage include a variety of standard and non-standard supplementary "Look" signs and placards as illustrated in Figure 27. Typically, where adequate median storage space is available, a "Look Right" sign or placard (similar to those

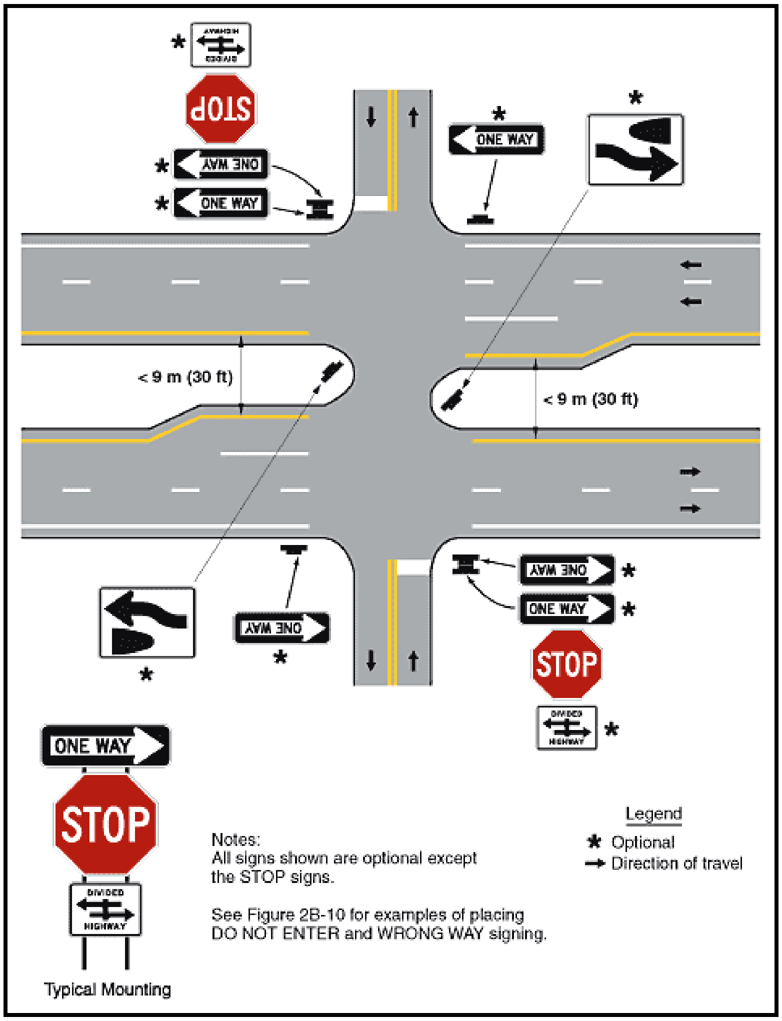


FIGURE 26. MUTCD Figure 2B-14; example of ONE WAY signing for divided highways with medians less than 30 feet (31)

shown in Figure 27A) is mounted beneath the STOP or YIELD signs within the median to advise minor road drivers to look right again for oncoming expressway traffic before leaving the median area; thereby conveying two-stage gap selection. When the median width does not allow for median vehicle storage, the standard regulatory LOOK sign (R15-8), meant for use at highway-rail grade crossings, and the standard CROSS TRAFFIC DOES NOT STOP warning plaque (W4-4p) shown in Figure 27B have been used in this application to remind minor road drivers to look both ways before crossing; thereby attempting to convey one-stage gap selection. Although no known scientific evaluation of the effectiveness of these signs has been conducted, several states are known to use them and believe they help reduce the occurrence of right-angle crashes.

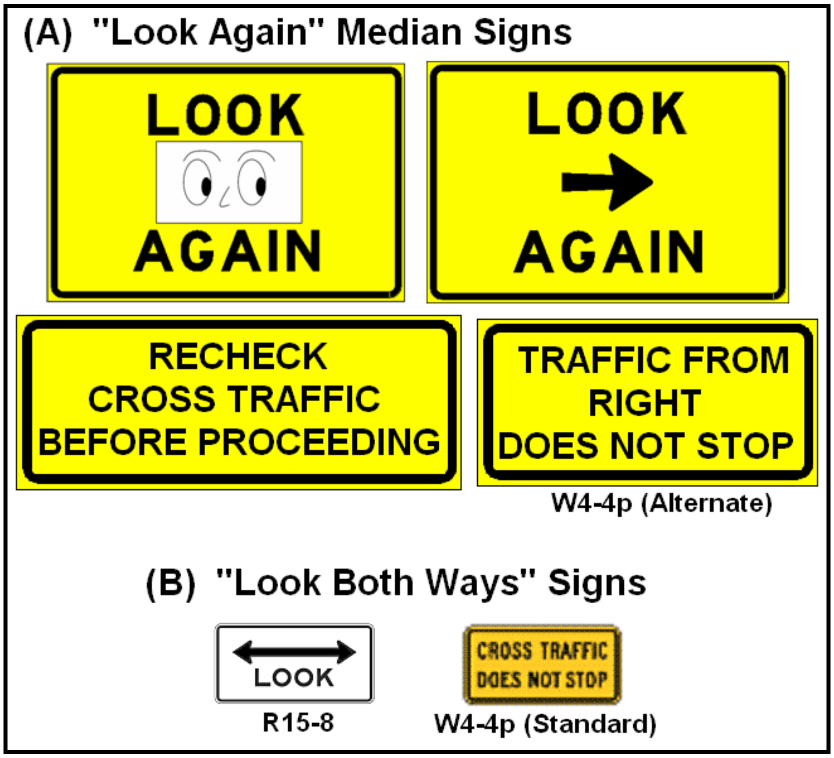


FIGURE 27. Warning signs used to prevent right-angle collisions

Two other median delineation treatments have been observed at rural expressway intersections. The first, illustrated in Figure 23, involves providing dashed pavement markings which extend the left interior edge line of the expressway through the intersection (i.e., across the median opening) to physically delineate the boundaries of the median area. In doing so, this treatment may give minor road drivers a better sense of how much storage space is available within the median; thus providing an enhanced visual cue for one or two-stage gap selection and minimizing through lane encroachments by vehicles stopped in the median. Although NCHRP 375 (*9*) states that this treatment should be used at intersections with median widths of 60 feet or less, it could potentially be used at any expressway intersection; however, its best usage may be at median crossovers where the median is too narrow to store a passenger car; thereby providing a visual cue to minor road drivers that one-stage gap selection is necessary. This treatment could also act as an intersection recognition device for approaching expressway drivers as it may enhance their ability to recognize the presence of an intersection (*16*). According to NCHRP 375 (*9*), this treatment has been used by at least two STAs; however, the effectiveness of this strategy in reducing crashes has not been quantified (*16*).

As discussed in the previous section, NCHRP 375 (*9*) found that opposing left-turn drivers leaving the expressway tend to turn in front of one another when the median width is 50 feet or less, but tend to turn behind one another when the median width is greater than 50 feet. The final median delineation treatment discovered includes the use of median islands (typically painted) with tubular delineators to channelize the median in order to guide opposing left-turn leaving expressway drivers into a "turn in front" behavioral path when the median width is greater than 50 feet. MoDOT uses this treatment for their Type II Median Opening which includes traditional left-turn lanes and a minimum median width of 60 feet as shown in Figure 28 (*37*). Photos of this median channelization are shown in Figure 29. Another benefit of this design is that the median islands provide space to more prominently display the median STOP or YIELD signs so that they lie more in the direct line-of-sight of the minor road driver as shown in the top portion of Figure 29. However, no studies on the safety effectiveness of this treatment have been conducted to date.

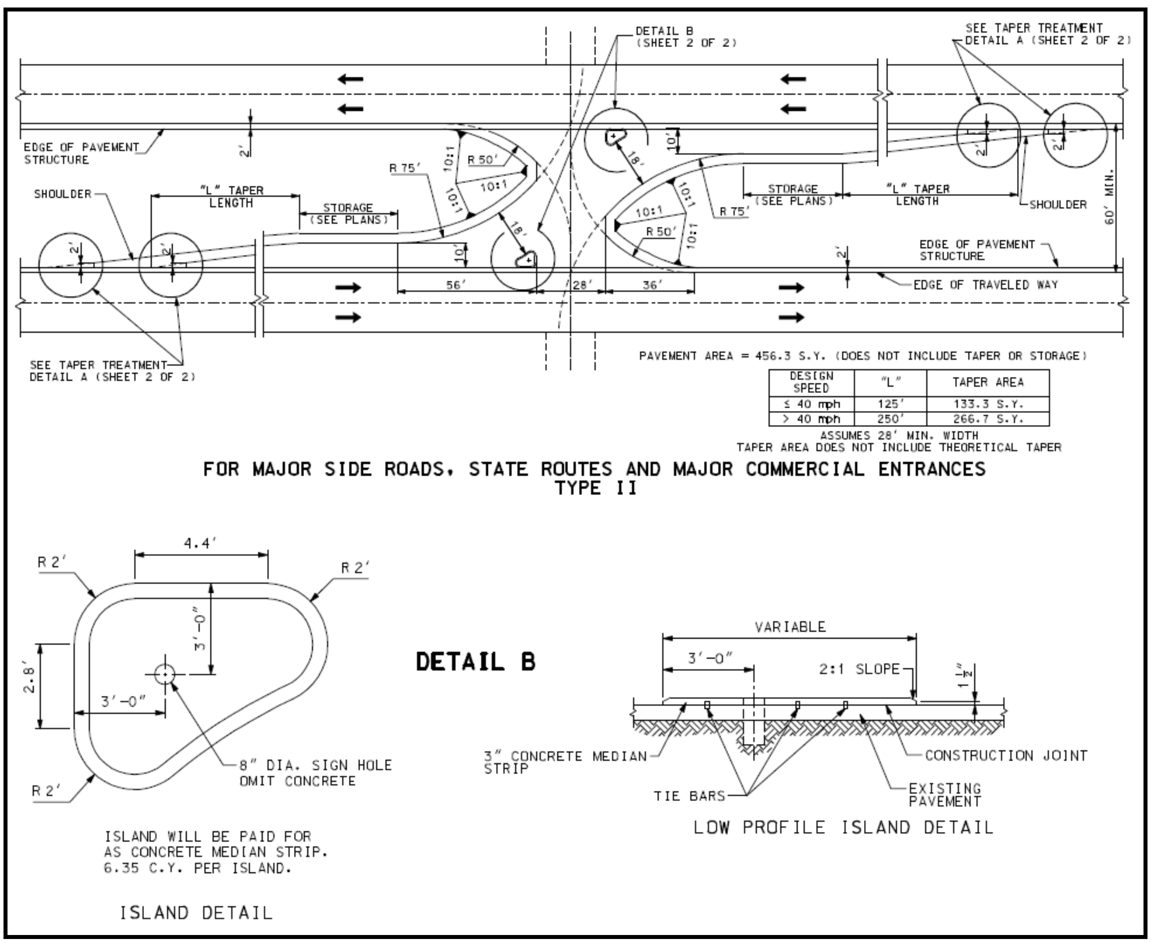


FIGURE 28. MoDOT Type II median opening plan with median channelization (37)

FIGURE 29. Photos of MoDOT Type II median opening with median channelization

2.3.3 Intersection Recognition Devices

Many TWSC rural expressway intersections are not readily visible to approaching drivers, particularly from the uncontrolled expressway approaches. As a result, crashes may occur because approaching expressway drivers are unaware of the intersection and are not prepared to deal with conflicts that may arise. Crashes may also occur because approaching minor road drivers do not recognize that they are approaching a stop-controlled intersection, which leads them to run the stop sign. Intersection recognition devices are commonly applied countermeasures such as intersection lighting, advance warning signs/beacons, advance guide signs, and rumble strips which enhance the visibility of intersections from at least one approach and thus, the ability of approaching drivers to perceive them. Providing greater intersection recognition reduces the likelihood that a minor road driver will run the stop sign and helps alert the expressway driver to proceed through the intersection with caution. FHWA's *Guidelines and Recommendations to Accommodate Older Drivers and Pedestrians* (*38*) encourages such improvements to enhance the driving environment for older drivers, and traditionally, when right-angle crashes begin to occur at TWSC rural expressway intersections, these treatments are the first countermeasures to be applied because they are relatively low-cost and easy to deploy. However, research has shown that lack of intersection recognition (i.e., stop sign violation) is the major contributing factor in only a very small fraction of right-angle crashes occurring at TWSC rural intersections (*4, 39*); thus, these treatments do not typically address the predominant cause of right-angle crashes which seems to be gap selection. Nevertheless, two intersection recognition devices for expressway drivers (freeway-style advance intersection guide signs and dynamic advance intersection warning systems) were selected for further study and are examined in detail as case studies within Chapter 3 of this thesis. Other intersection recognition devices are briefly introduced over the remainder of this chapter.

2.3.3.1 TWSC Beacons

TWSC beacons, also known as intersection control beacons (ICBs) or bouncing ball beacons (BBBs), are typically suspended over an intersection with flashing yellow indications to the expressway approaches and flashing red indications to the minor road approaches in order to enhance approaching driver awareness of an intersection and reinforce the assignment of right-of-way at the intersection. Section 4K.02 of the MUTCD (*31*) briefly addresses warrants for the installation of ICBs by stating, "Intersection control beacons may be used at intersections where traffic or physical conditions do not justify conventional traffic control signals, but crash rates indicate the possibility of a special need." In 1991, Hall (*40*) developed general guidelines/recommended warrants for the installation of ICBs at rural intersections for the New Mexico State Highway and Transportation Department. The suggested warrants are primarily based on two-year crash experience and sight distance limitations.

According to a 1993 survey conducted by Bonneson et al. (*27*), 9 of the 23 responding STAs indicated that they had installed this type of beacon as a corrective measure at TWSC divided highway intersections with high crash rates. However, some STAs (Mn/DOT for example) only install ICBs above intersections with all-way stop control due to the fact that minor road traffic may be confused regarding the nature of control on the mainline (i.e., a minor road driver may incorrectly assume that the mainline approaches are also controlled by a red flasher) (*41*). As an alternative to ICBs, Mn/DOT chooses to install flashing red beacons above the stop signs on the minor road approaches and flashing yellow warning beacons above "Intersection Ahead" signs on the mainline approaches. Other STAs supplement the overhead ICBs with the "CROSS TRAFFIC DOES NOT STOP" (W4-4p) placard shown in Figure 27 mounted below the minor road stop signs. In either case, these beacons are expected to help reduce right-angle and night-time crashes related to stop sign violations on the minor road approaches and lack of intersection awareness on the part of expressway drivers.

Research on the effectiveness of ICBs in reducing the frequency and/or severity of collisions at intersections specifically on divided highways is very scarce as only one such study was found. In 1959, Solomon (*17*) showed that the installation of an ICB significantly reduced both crash frequency and severity at five four-legged divided highway intersections in Michigan. On the contrary, several studies have evaluated the safety effects of ICBs at TWSC intersections on two-lane undivided highways or at intersections where the roadway type was not specified (*17, 19, 42, 43, 44*). Overall, these studies have found mixed results.

In 1992, the operational effects of ICBs at divided highway intersections were investigated by Pant et al. (*44*). The study compared stop-sign violations, delay, approach speeds, and accepted gap sizes at two divided highway intersections with ICBs versus two divided highway intersections without ICBs and found that, in the presence of ICBs: 1) the percentage of rolling stops was reduced, particularly at night, 2) delays increased by approximately 3.0 seconds per vehicle, 3) mean approach speeds were significantly reduced on all intersection approaches with a corresponding reduction in 85th percentile speeds and speed variance, and 4) the size of accepted gaps during the daytime was significantly reduced for all minor road movements into or through the near-side expressway traffic stream. However, it is speculated that the effectiveness of ICBs is related to their relative uniqueness and they should not be overused (*16, 40, 44*).

2.3.3.2 Intersection Lighting

According to NCHRP 500, Volume 5 (*16*), improving the visibility of unsignalized intersections by providing lighting at the intersection itself or on its approaches is a proven strategy for reducing nighttime crashes; meaning properly designed evaluations have been conducted which show this strategy to be effective. The major evaluation referenced in drawing this conclusion was a 1999 study conducted by Preston and Schoenecker (*45*). In this study, a comparative analysis of 3,495 isolated, rural, two-lane, through-stop intersections (259 with lighting and 3,236 without lighting) revealed that intersections with lighting had a 25 percent lower nighttime crash rate as compared to intersections without lighting which was a statistically significant difference at a 99.5 percent level of confidence. In the same study, a before-after crash analysis of 12 rural intersections where lighting was installed showed a 40 percent reduction in the nighttime crash rate (statistically significant at a 95 percent level of confidence), a 20 percent reduction in nighttime fatal and injury crashes (statistically significant at a 90 percent level of confidence), and a 44 percent reduction in the nighttime right-angle crash rate (not statistically significant at a 90 percent level of confidence). Preston and Schoenecker also conducted a benefit-cost analysis based on these results and found that the crash reduction benefits associated with the installation of intersection lighting at rural intersections outweigh the costs by a wide margin (the average benefit-cost ratio was approximately 15 to 1). In 2006, Isebrands et al. (*46*) conducted a follow-up study on intersection lighting which included a larger sample of rural intersections in Minnesota. Before-after analysis of 48 rural through-stop intersections where lighting had been installed showed that the nighttime crash rate was reduced by 19 percent while the daytime crash rate increased by 26 percent. This study also revealed an 11 percent decrease in nighttime crash severity while the daytime crash severity increased by 30 percent. Further analysis of 33 rural through-stop intersections where lighting had been installed with three years of before and after crash data indicated a 59 percent reduction in nighttime crash rate which was statistically significant at a 90 percent level of confidence.

Given the apparent difficulty with gap selection at divided highway intersections, it is reasonable to believe that the installation of intersection lighting would have similar nighttime safety benefits at these locations. However, only one study was found which examined the safety effects of intersection lighting specifically at divided highway intersections. In 1995, NCHRP 375 (*9*) developed separate SPFs for 153 four-legged and 157 three-legged through-stop divided highway intersections in rural California. Both models developed included many variables, but suggested that the presence of intersection lighting was a significant factor which unexpectedly increased intersection crash frequency by approximately 37 and 133 percent at four and three-legged intersections, respectively; however, this research did not separately examine collisions which occurred during night versus day. Therefore, further research is required to quantify the safety effects of lighting installations at rural expressway intersections and to determine how the quantity and location of luminaires impacts the safety of these intersections.

Furthermore, guidelines/warrants for the installation of lighting at rural expressway intersections should be developed. Preston and Schoenecker (*45*) and Isebrands et al. (*46*) examined existing national and state agency guidelines for the installation of lighting at rural intersections in 1999 and 2006, respectively. They found that most existing guidelines are based on nighttime traffic volumes and crash frequencies. Preston and Schoenecker concluded that Mn/DOT warrants for intersection lighting were too stringent and recommended that Mn/DOT reduce their volume and crash experience thresholds in order to encourage the installation of intersection lighting at more rural intersections.

2.3.3.3 Minor Road In-Lane Rumble Strips

Rumble strips are raised or grooved transverse patterns constructed on the roadway surface which are intended to provide drivers with a tactile vibration and an audible warning that they need to be alert to the driving task. In-lane rumble strips can be installed on intersection approaches to call the approaching drivers' attention to the presence of the intersection and the traffic control in place at the intersection. They are particularly appropriate on stop-controlled intersection approaches where a pattern of "ran the stop sign" crashes or stop sign violations exist due to lack of driver recognition of the stop control (*16*). There are two types of in-lane rumble strips: those that cross the entire width of the approach lane as shown in the upper portion of Figure 30 and those that cross only the wheel paths as shown in the lower portion of Figure 30 (*47*). In 2001, Harder et al. (*48*) found that drivers brake earlier and harder in the presence of full width rumble strips than they do with wheel path rumble strips; however, these results were obtained using a driver simulator under daylight conditions for drivers that were alert.

According to a 1993 survey conducted by Harwood (*49*), 41 STAs had installed rumble strips in the traveled way and 37 of those had placed them on approaches to intersections. In 2004, Maze et al. (*2*) surveyed 28 STAs and 12 reported using in-lane

FIGURE 30. In-lane intersection approach rumble strips (47)

rumble strips on the minor road stop-controlled approaches to expressway intersections. However, the safety benefits of in-lane rumble strips on intersection approaches have not been precisely quantified as NCHRP 500, Volume 5 (*16*) categorizes this treatment as a tried strategy for which valid evaluations have not been conducted. Despite the lack of conclusive findings regarding in-lane rumble strips on intersection approaches, Harwood (*49*) indicates that they can provide a 50 percent reduction in rear-end and “ran the stop sign” collisions; however, they should be used sparingly so that their surprise value in gaining the driver’s attention is retained. In addition, they create excessive noise which can negatively impact nearby homes and businesses. Therefore, in-lane approach rumble strips are only recommended after other measures, such as “STOP AHEAD” signs, markings, or flashers, have failed to correct the crash pattern (*16, 47, 49*).

2.3.3.4 Minor Road Splitter Islands

Another intersection recognition device which may be used on stop-controlled minor road approaches to call the approaching driver’s attention to the presence of the intersection and the stop control is to construct “splitter” or divisional islands at the mouth of the intersection. A splitter island refers to a channelizing island that separates opposing traffic as shown in Figure 31 (*50*). These islands, combined with edge line striping that narrows the lane width in the intersection’s throat, provide additional space to mount a second stop sign and are generally believed to be effective in declaring the presence of an intersection, reducing minor road approach speeds, increasing stop sign compliance, guiding minor road traffic through the intersection, and improving intersection safety. However, little research exists which examines their effectiveness in these areas. According to NCHRP 500, Volume 5 (*16*), this strategy is still unproven, but is more appropriate on minor road approaches to skewed intersections or on minor road approaches where the approach speeds are high.

FIGURE 31. Splitter island on minor road approach to an expressway intersection (50)

Another potential benefit of splitter islands, particularly at high-speed rural expressway intersections, may be that their presence reduces the speed of right-turn leaving expressway traffic and helps protect/shield minor road traffic waiting at the stop bar from being stuck by right-turn leaving expressway vehicles that would otherwise make wider, higher-speed turns. However, no research has been conducted to examine this possible advantage.

2.4 CHAPTER 2 SUMMARY

Through a literature review, this chapter examined the safety effects of various rural expressway intersection features (geometric design elements and traffic control/operational elements) and other rural expressway intersection safety treatments that are not addressed in the Chapter 3 case studies, but have been experimented with by STAs. Rural expressway intersection safety treatments can be divided into 3 broad categories: conflict point management techniques, gap selection aids, and intersection recognition devices. These treatment categories were defined and popular treatments from each of these categories were examined in this chapter.

CHAPTER 3: CASE STUDIES OF TEN RURAL EXPRESSWAY INTERSECTION TREATMENTS

3.1 OVERVIEW

As described in the previous chapter, STAs have experimented with a wide range of intersection safety treatments at problematic rural expressway intersections. Unfortunately, literature on many of these treatments is scarce; thus, their safety effectiveness is relatively unknown and national geometric design guidance is lacking. Therefore, "case studies" for ten of the most promising rural expressway intersection safety strategies identified in the literature review were conducted to investigate and document the experience of STAs who have implemented these countermeasures. After it was determined which STAs had implemented the strategies of interest, knowledgeable staff from the respective agencies were interviewed to determine: 1) the circumstances surrounding the treatment's implementation (i.e., reasons for, cost, etc.), 2) intersection site conditions (i.e., type and intensity of land use, traffic volumes and patterns, geometry including horizontal/vertical alignment, etc.), 3) public reaction (i.e., complaints, elderly driver issues, indications of erratic driver behavior, etc.), 4) any lessons learned including design guidance and general advice, and 5) if the agency had performed any subjective or objective evaluations of the operational and/or safety performance of the treatment. If no safety assessments had been performed, before and after crash data were obtained from the agency, where possible, for the purpose of conducting naïve before-after safety analysis of each treatment. The limitations of the naïve before-after analysis methodology have been well documented (*22*) and by no means are the before-after analyses reported in the case studies meant to be scientifically rigorous evaluations which develop reliable crash reduction factors. Instead, they are simply observational before-after studies which compare the count of crashes in the before period with the count of crashes in the after period to try to begin to understand each treatment's potential for improving rural expressway intersection safety.

Ten case studies are included in this chapter. They include three conflict point management techniques (J-turn intersections, offset T-intersections, and jughandle intersections), five gap selection aids (IDS technology, static roadside markers, MALs, offset right-turn lanes, and offset left-turn lanes), and two intersection recognition devices for expressway drivers (freeway-style advance intersection guide signs and dynamic advance intersection warning systems). No before-after crash data were available for three of the ten treatments investigated (jughandle intersections, IDS technology, and static roadside markers). In each of the other seven case studies, a limited number of sites were examined and, in most instances, the amount of before and after crash data were inadequate to draw any solid conclusions; however, where more than three years of before and after data were obtained, statistical evaluations were performed. Nevertheless, these naïve before-after evaluations remain flawed because 1) they do not take regression-to-the-mean into account and 2) it is not known what part of the noted change in safety can actually be attributed to the treatment and what part may be due to changes in other external factors (volume, weather, driver demographics, etc.).

3.2 J-TURN INTERSECTION CASE STUDY

3.2.1 Description

The ability to accommodate high volumes of traffic safely and efficiently through intersections largely depends on the arrangements provided for handling intersecting traffic (*3*). All movements through a typical TWSC rural expressway intersection do not have the same crash risk. The highest risk movements (i.e., those accounting for the largest share of severe crashes) tend to be minor road maneuvers through the far-side intersection (i.e., minor road left-turn and crossing maneuvers) (*4, 14*). Therefore, elimination of these maneuvers and their associated conflict points can be an effective means of improving safety at rural expressway intersections. An intersection design which accomplishes this is the “J-turn” intersection shown in Figure 32. The term “J-turn” for this style of intersection was coined by the Maryland State Highway Administration (MSHA); however, this intersection design has also been known by other names in other states, such as the “Superstreet” intersection in North Carolina or the “Right-Turn U-Turn” (RTUT) intersection in Florida.

The J-turn intersection combines a directional median opening (which allows direct left-turn exits from the expressway, but prohibits minor road traffic from entering the median) with downstream median U-turns. As a result, minor road traffic wishing to turn left or cross straight through the intersection are forced to make these maneuvers indirectly by turning right, weaving to the left, making a downstream U-turn, and then returning to the intersection to complete their desired maneuver. There is no indication that U-turns at unsignalized median openings constitute a safety concern (*28*). Therefore, the J-turn intersection design replaces the high risk, far-side conflict points associated with direct minor road left-turns and crossing maneuvers (i.e., conflict points 15, 16, 19, 21, 22, and 25 in Figure 2) with less risky conflict points associated with right-turns, U-turns, and weaving maneuvers. Overall, the J-turn intersection reduces the total number of intersection conflict points from 42 at a typical TWSC rural expressway intersection as shown in Figure 2 to 24 as shown in Figure 32. Not only are the total number of conflict points reduced, but more importantly, the J-turn intersection eliminates 20 crossing path conflict points present at a typical TWSC rural expressway intersection, thereby reducing the opportunity for right-angle/broadside collisions.

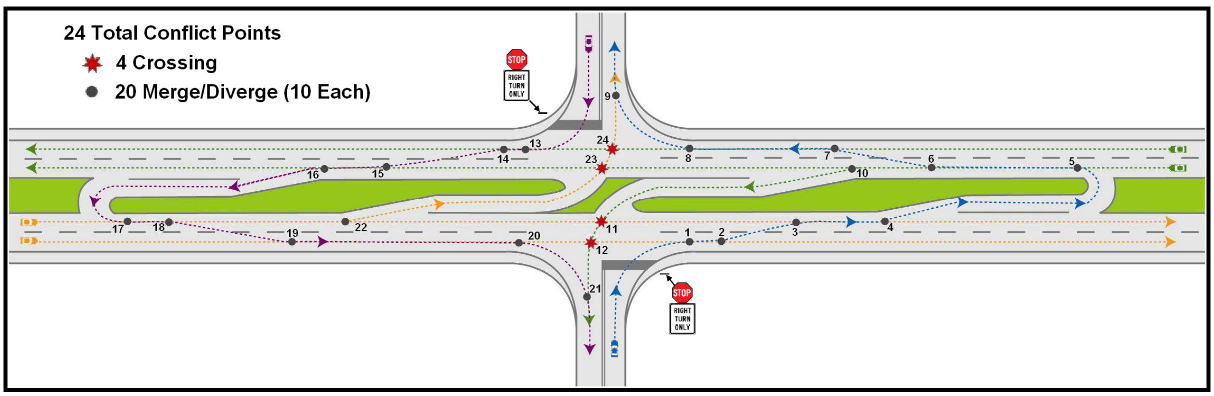


FIGURE 32. J-turn intersection schematic & conflict point diagram

3.2.2 Existing Design Guidance

The J-turn intersection is one possible countermeasure between a typical TWSC rural expressway intersection and an interchange which still allows a reasonable level of accessibility to drivers on the minor road. Variations of this countermeasure (such as the median U-turn intersection described in Chapter 2) have been used previously in Michigan and Florida, but often in urban and suburban areas at signalized intersections. The use of the J-turn intersection design at high-speed TWSC rural expressway intersections is a more recent application; therefore, national design guidance for the J-turn intersection is relatively non-existent. In fact, this type of intersection is only briefly mentioned in Chapter 9 of the AASHTO Green Book (*3*).

On page 709, the Green Book (*3*) currently discourages the use of a J-turn type intersection on high-speed or high volume highways due to "the difficulty of weaving and the long lengths involved" in the indirect minor road movements, unless "the volumes intercepted are light and the median is of adequate width." Green Book Exhibit 9-92 (Figure 14) provides minimum median widths to accommodate U-turns by different design vehicles turning from the inside (passing) lane of a four-lane divided facility to various locations (i.e., inner lane, outer lane, or shoulder) in the opposite direction. However, on page 710, the Green Book (*3*) states:

> *U-turn openings designed specifically for the purpose of eliminating the left-turn movement at a major intersection should be designed with a median left-turn lane for storage. If the U-turn is made from a median deceleration lane, the total median width required would include an additional 12 feet for a single median turn lane.*

In addition, on a high-speed expressway, it would be ideal if the median width was wide enough to allow the design vehicle to U-turn into a median acceleration lane in the opposite direction, thereby minimizing interference with high-speed expressway traffic. Therefore, Table 8 was developed using Green Book Exhibit 9-92 as a basis to show the minimum median widths required to implement a J-turn intersection. As Table 8 shows, the minimum median width varies from 20 to 95 feet depending on the design vehicle and desired U-turn maneuver.

TABLE 8. Minimum Median Width (Feet) for J-Turn Intersection U-Turns

TYPE OF MANEUVER	DESIGN VEHICLE		
U-Turn From Deceleration Lane Into…	**19 ft. P**	**30 ft. SU 40 ft. BUS**	**55 ft. WB-50 65 ft. WB-60**
Acceleration Lane	54	87	95
Inner Lane	42	75	83
Outer Lane	30	63	71
Outside Shoulder	20	53	61

NOTE: Median width is the dimension between the edges of opposing through lanes and includes left shoulders as well as median deceleration/ acceleration lanes. 12 foot wide lanes have been assumed.

According to page 457 of the Green Book, in rural areas, "The school bus is often the largest vehicle to use the median roadway frequently." Therefore, where a school bus is selected as the design vehicle and a median width of at least 63 feet cannot be provided, according to Table 8, a loon (an expanded paved apron opposite a U-turn crossover as illustrated in Figure 33) or a left-hand U-turn jughandle (shown in Figure 34B) should be considered to accommodate the larger U-turning path. A right-hand U-turn jughandle (shown in Figure 34A) should not be used in conjunction with a J-turn intersection as it would defeat the purpose of the J-turn by recreating the crossing path conflict points associated with minor road left-turn maneuvers. NCHRP 524 (*28*) examined several unsignalized median openings with loons and, although the sample size was limited, found no indication that the provision of loons or their use by large trucks leads to safety problems. In 2003, Sisiopiku and Aylsworth-Bonzelet (*51*) evaluated the design and operation of existing loons and developed guidelines for loon design as shown in Table 9.

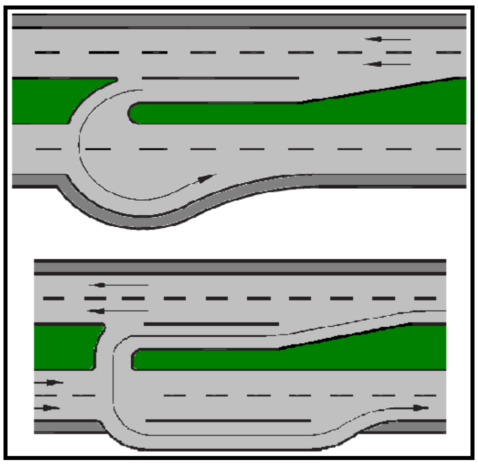


FIGURE 33. Examples of U-turn median openings with left-turn lanes & loons

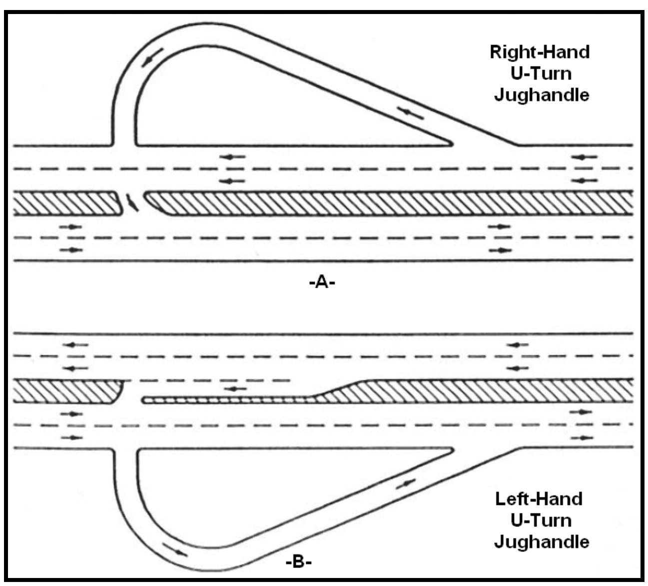


FIGURE 34. Green Book Exhibit 9-93; special indirect U-turns with narrow medians (3)

TABLE 9. Recommended Loon Width (Feet) on Four-Lane Divided Roadways (*51*)

	DESIGN VEHICLE		
Median Width (ft)	**19 ft. P**	**30 ft. SU 40 ft. BUS**	**55 ft. WB-50 65 ft. WB-60**
0	16.40	49.21	59.06
16.40	0	32.81	42.65
32.81	0	16.40	26.25
49.21	0	0	9.84
65.62	0	0	0

NOTES: Loon width equal to zero indicates that standard shoulder width is sufficient. Dimensions converted from metric units as provided in original report.

A final design issue related to the J-turn intersection is the spacing between the main intersection and the median U-turns. Ultimately, the selection of the most appropriate separation distance is a trade-off between providing sufficient space for safe/functional weaving areas as well as adequate U-turn storage (i.e., to minimize spillback potential), while minimizing the travel distance/time of the indirect left-turn and crossing maneuvers. For a signalized median U-turn intersection as described in Chapter 2, the Green Book (*3*) recommends that the U-turn crossovers be located 400 to 600 feet from the main intersection, or midblock between adjacent intersections. However, no national guidance is provided for an unsignalized high-speed application and the safety impact of the separation distance in rural areas is still unclear. Existing research regarding indirect left-turns via RTUTs has been exclusively conducted in urban and/or suburban settings. One such study conducted by Liu et al. (*52*) found that the majority of crashes related to RTUTs in Florida occur in the weaving areas rather than directly at the U-turn locations and the frequency of these collisions is significantly impacted by mainline volumes and the separation distance. The SPF developed through this research showed that a 10 percent increase in separation distance resulted in a 4.5 percent decrease in weaving area collisions. However, more research needs to be conducted to determine the optimum U-turn spacing for a J-turn intersection located on a rural expressway.

J-turn intersections have already been constructed on rural expressways in Maryland, North Carolina, Missouri, and Florida; while other states like Iowa and Minnesota are seriously considering their use on rural expressways. Some state design guidance on J-turns is available within the North Carolina Department of Transportation (NCDOT) Roadway

Design Manual (*53*), the MoDOT Engineering Policy Guide (*37*), and the Florida Department of Transportation (FDOT) Design Standards (*54*). The NCDOT and MoDOT standard geometric plans for J-turn intersections are shown in Figures 35 and 36, respectively. MoDOT's standard J-turn intersection signing plan is shown in Figure 37. According to NCDOT design policy, the median U-turn should be located downstream approximately 800 to1000 feet (*53*). Similarly, MoDOT policy states that the U-turn should be located approximately 600 to 1000 feet downstream, with the specific location to be determined via capacity analysis software (*37*). The MSHA has successfully used longer separation distances in the range of 1,500 to 2,500 feet. Although FDOT Design Standards (*54*) do not recommend a U-turn offset or contain a standard plan for a full J-turn intersection, they do include standard plans for directional median openings with either parallel or tapered offset left-turn lanes as shown in Figure 38.

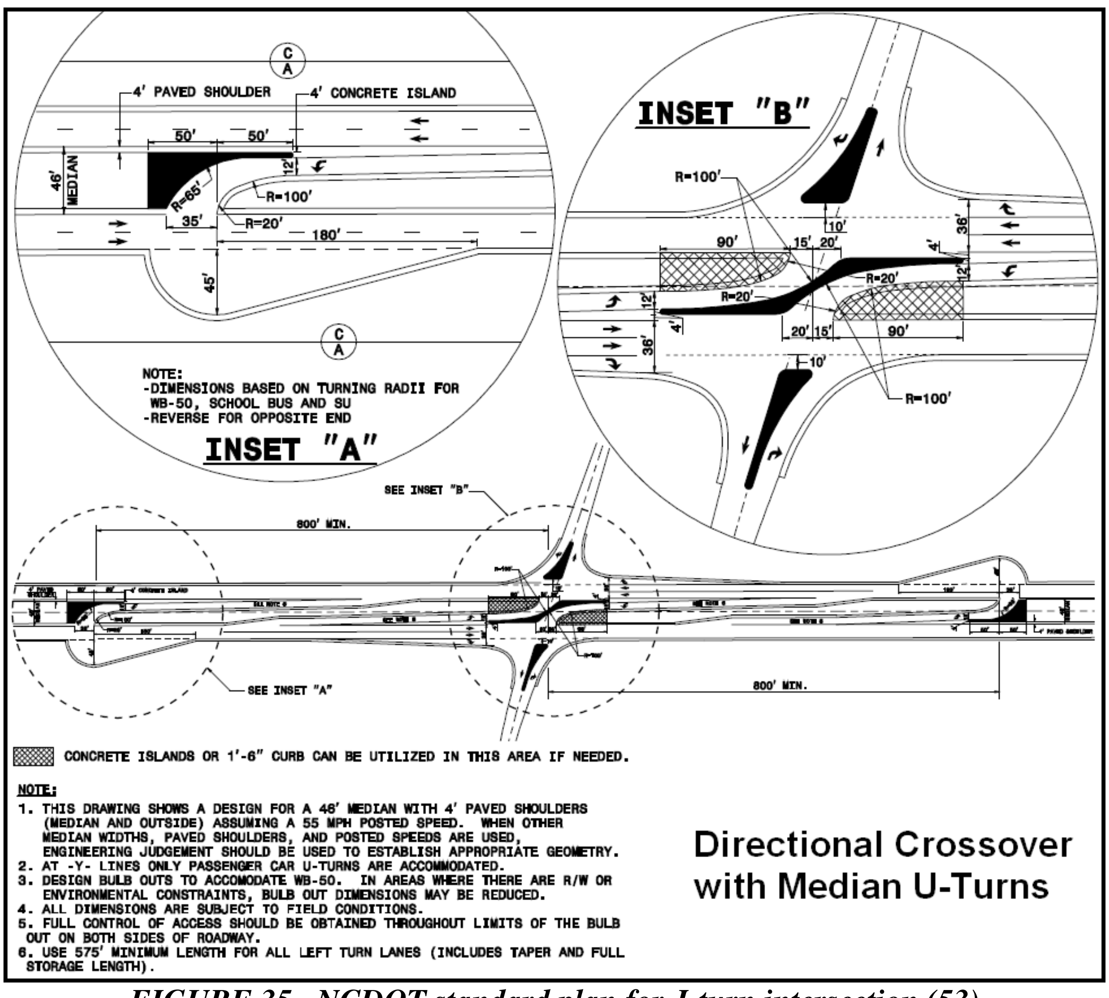


FIGURE 35. NCDOT standard plan for J-turn intersection (53)

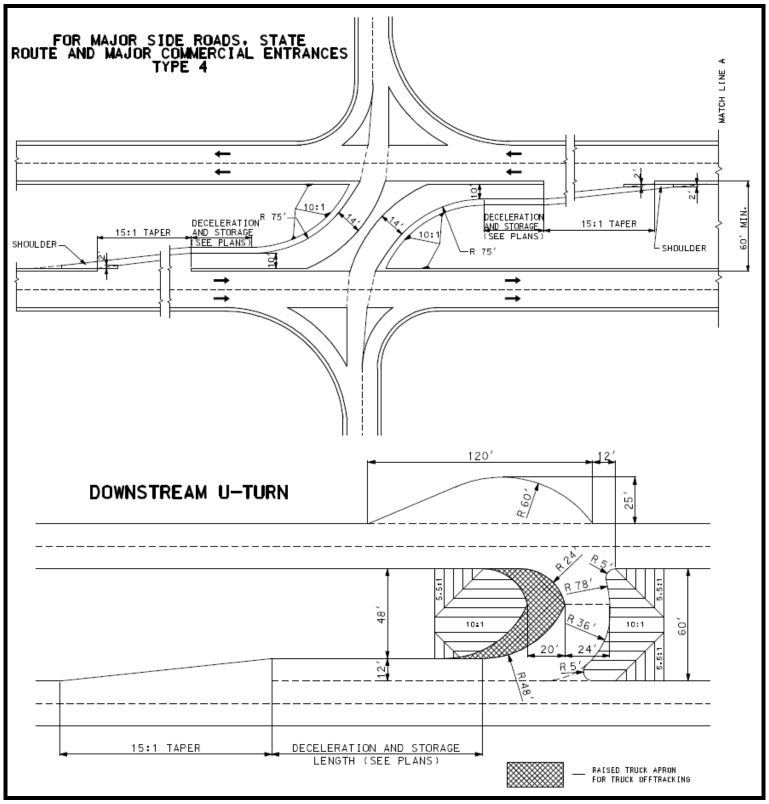


FIGURE 36. MoDOT standard plan for J-turn intersection (37)

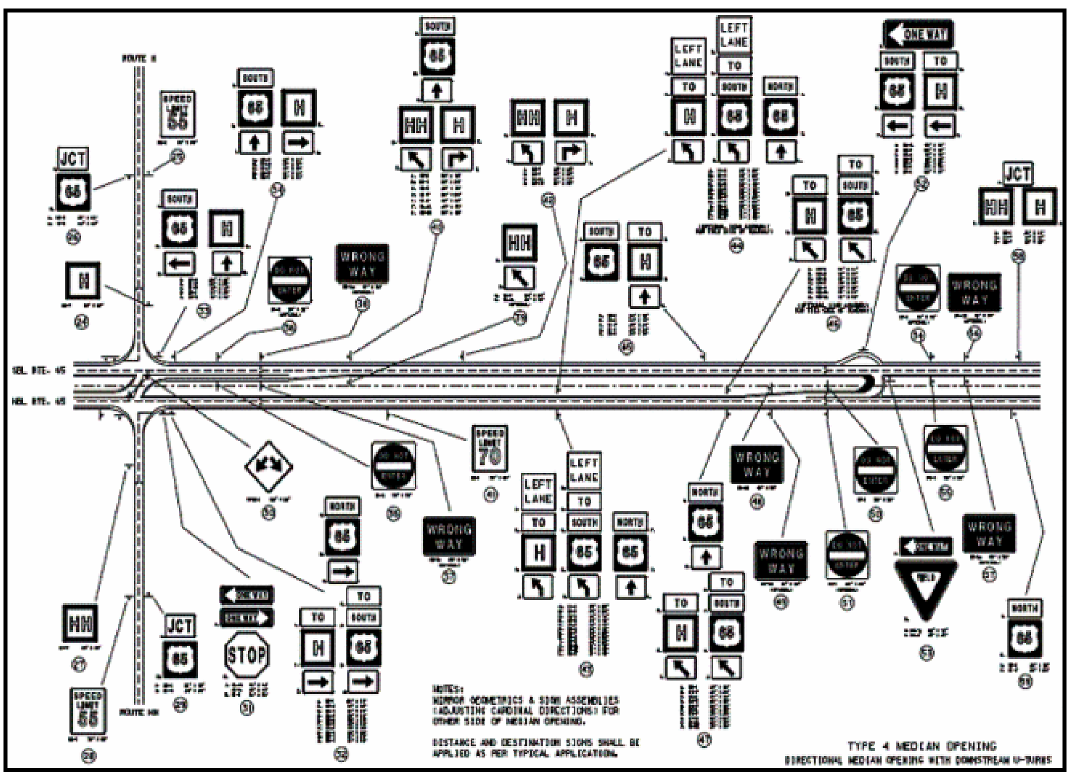


FIGURE 37. MoDOT standard signing plan for J-turn intersection (37)

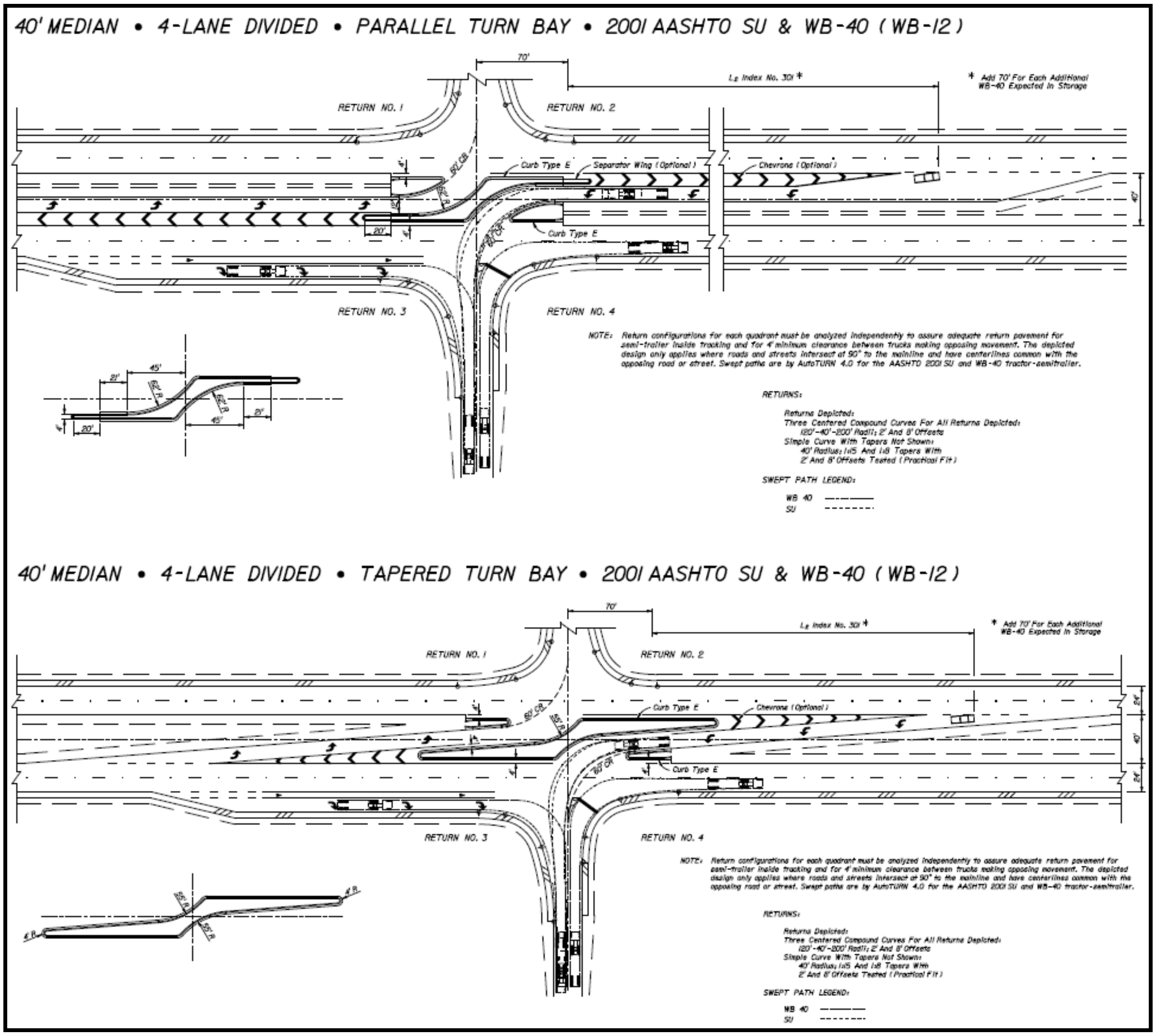


FIGURE 38. FDOT standard plans for directional median openings (54)

3.2.3 Case Studies of Implementation & Safety Effectiveness

For this case study, the experiences of the MSHA and the NCDOT with J-turn intersections on high-speed rural expressways were examined. Their experiences are described herein.

3.2.3.1 Maryland Experience

The MSHA has been constructing J-turn intersections as a safety countermeasure at high-speed rural expressway intersections since November of 2000 when they converted the intersection of US-301 and Maryland State Highway 313 (MD-313) in Kent County, Maryland from a traditional TWSC expressway intersection into a J-turn intersection. An aerial photo of this J-turn intersection is provided in Figure 39 and a more detailed intersection schematic is shown in Figure 40 (*55*). At the intersection, US-301 is a four-lane divided expressway functionally classified as a rural principal arterial which has partial access control, a 60 foot wide median, and a posted speed limit of 55 mph. MD-313 is an undivided highway functionally classified as a rural major collector having no access control and a posted speed limit of 40 mph. The 2000 ADT for US-301 in the vicinity of the intersection was approximately 10,600 vpd while MD-313 had an ADT of approximately 1,450 vpd (*56*). Peak hour movements recorded in March of 1999 are shown in Table 10.

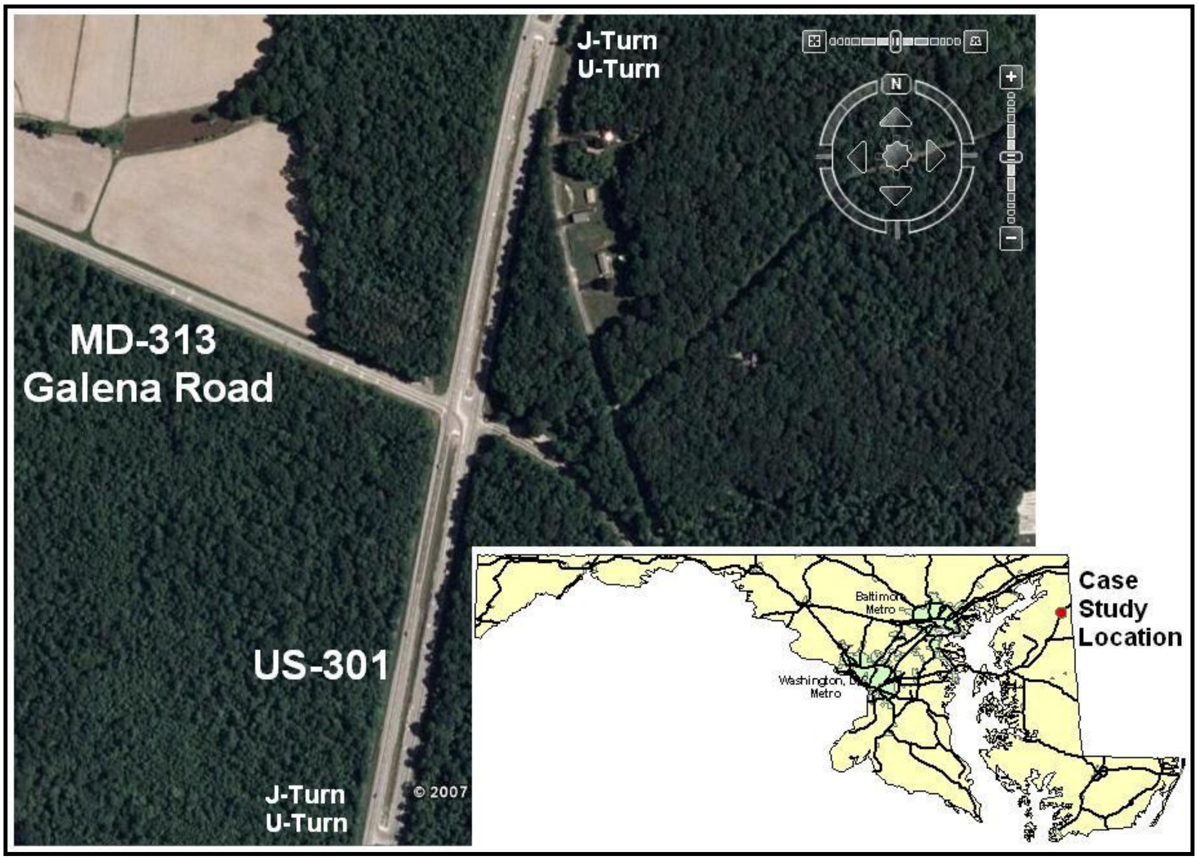


FIGURE 39. Aerial photo of J-turn intersection at US-301 & MD-313

TABLE 10. US-301 & MD-313 Peak Hour Movement Volumes on March 11, 1999

	US-301, SB			**US-301, NB**			**MD-313, EB**			**MD-313, WB**		
	Left	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**	**Left**	**Thru**	**Right**
AM Peak Hour	0	176	1	55	208	5	2	66	63	9	49	4
PM Peak Hour	6	319	6	51	220	9	2	54	60	4	41	5

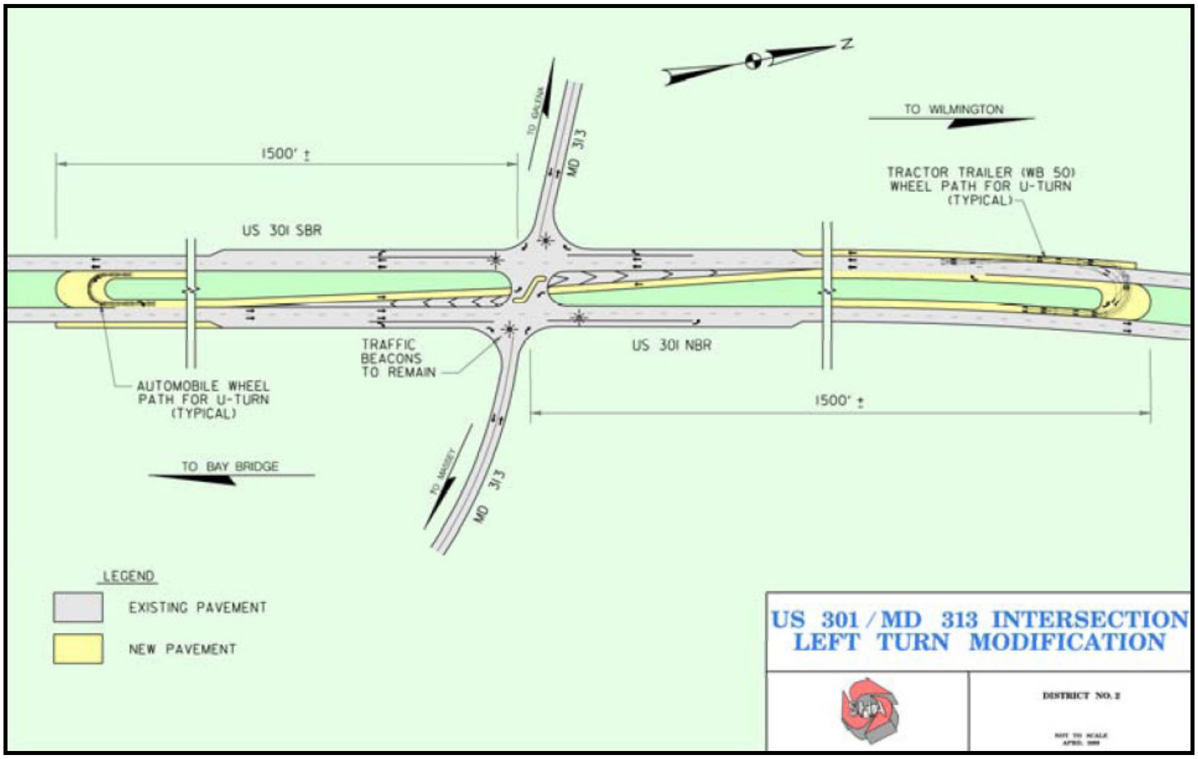


FIGURE 40. MSHA plan for J-turn intersection at US-301 & MD-313 (55)

Prior to construction of the J-turn intersection, intersection lighting was in place and located in the northeast and southwest quadrants of the intersection. The traffic control at the intersection included STOP signs on the MD-313 approaches (with YIELD signs in the median) along with overhead flashing beacons which flashed red toward the MD-313 approaches and yellow toward the US-301 approaches. The intersection lighting and beacons remained in operation after the conversion to the J-turn intersection took place. The stop control on MD-313 also remained in place after conversion with the median YIELD signs relocated to face left-turning traffic exiting US-301. Other traditional signage in place at the intersection included advance junction route assemblies, route confirmation assemblies, ONE WAY signs mounted above the STOP signs and in the median, divided highway signs mounted below the STOP signs, DO NOT ENTER signs on the upstream approaches of US-301, and NO PARKING signs located along US-301.

Between 1997 and 2000 (prior to construction of the J-turn intersection) there were a total of 33 crashes at the intersection (8.25 crashes per year), of which 1 resulted in a fatality (3 percent), 22 involved injuries (67 percent), and 10 involved property damage only (30 percent), giving an overall average crash rate of approximately 1.86 crashes per million entering vehicles (mev). Using the SPF developed by Maze et al. (*2*) for Iowa expressway intersections given in Table 3, an annual crash frequency of 3.21 crashes per year would be expected for an expressway intersection with similar traffic volumes. Therefore, this intersection's annual crash frequency was roughly 2.5 times (157 percent) higher than expected over this four year period. A collision diagram for this intersection during the before period is shown in Figure 41 (*57*). An examination of crash types reveals that the overwhelming majority (85 percent) of crashes occurring at this intersection in the before period could be considered "preventable" by the J-turn intersection configuration. Furthermore, 22 of the 33 collisions (67 percent) were right-angle collisions, 18 of which occurred on the far-side, accounting for 82 percent of all right-angle crashes. Weather conditions and darkness seemed to play a very small role in these collisions and there was no indication that sight distance was an issue at the intersection. Therefore, it is reasonable to suggest that the primary contributing factor to these crashes was related to gap recognition and selection by drivers on MD-313 attempting to cross or turn onto US-301.

Over the years, the MSHA had installed additional signage at the intersection to address its historically poor safety record. Two types of advance intersection signs were installed on each of the US-301 approaches and another type was placed on each of the MD-313 approaches. For a driver approaching the intersection on US-301, the first of these additional signs encountered was an intersection ahead warning sign (shown in Figure 42A) with additional text warning of cross traffic ahead at the flasher. This sign was placed on the right shoulder as well as within the median on each US-301 approach. In conjunction, the freeway style advance route guide sign shown in Figure 42B was also placed on the US-301 approaches to alert drivers of the upcoming intersection. Furthermore, the STOP AHEAD warning sign shown in Figure 42C was placed overhead on both MD-313 approaches to alert drivers to the presence of the stop control and the divided highway ahead. However, these signs failed to improve the safety performance of the intersection.

MSHA officials then planned to address the safety issues at this location by constructing an interchange. However, funding constraints forced them to cancel the interchange project. When this occurred, concerned local citizens demanded that the MSHA do something to address the high number of crashes at the intersection. As a result, the MSHA developed the lower cost J-turn intersection design strategy which directly addressed the far-side right-angle crashes which were occurring. The J-turn intersection conversion at US-301 and MD-313 illustrated in Figure 40 was completed in November of 2000 at a cost of approximately $618,000.

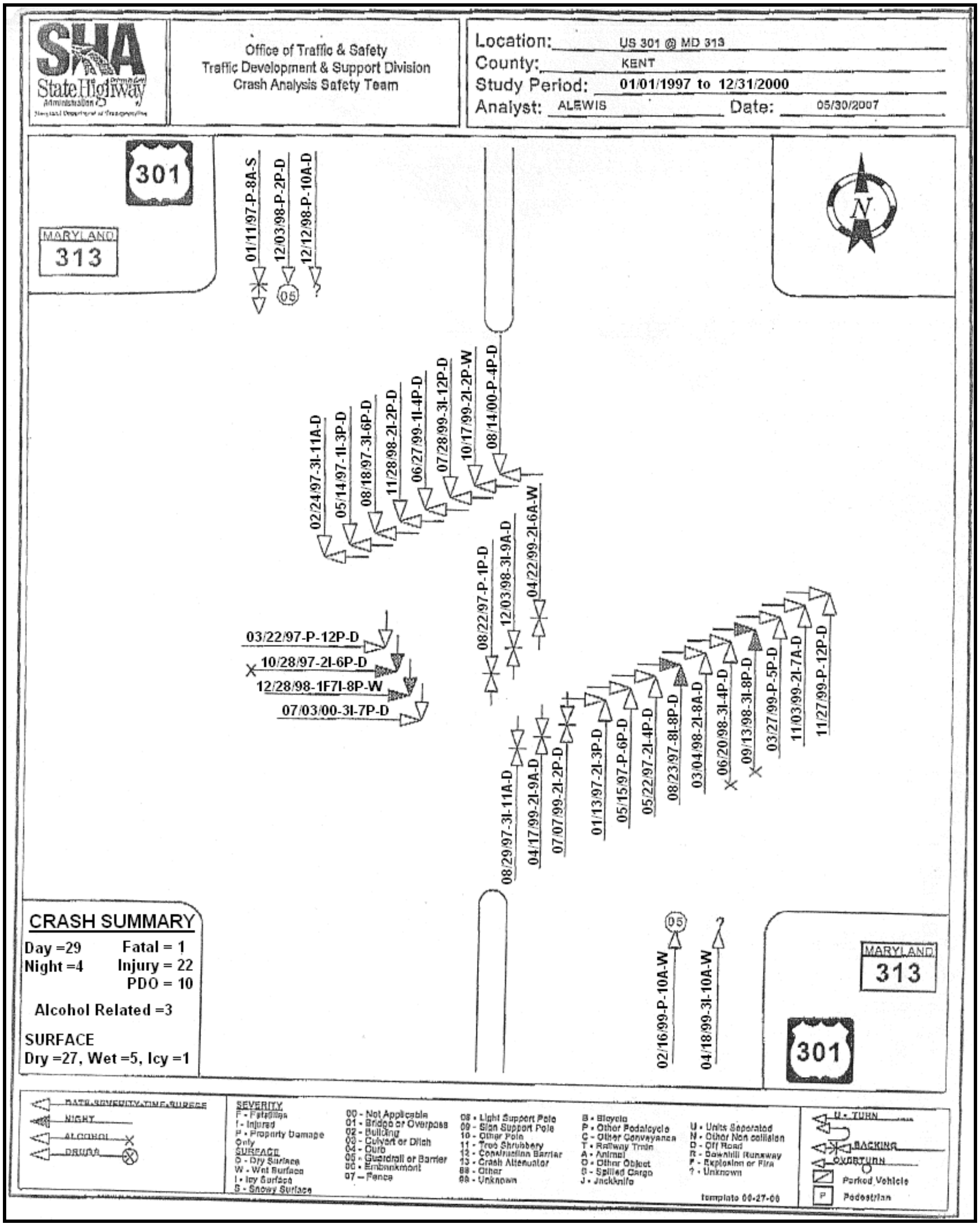


FIGURE 41. Collision diagram at US-301 & MD-313 before J-turn construction (57)

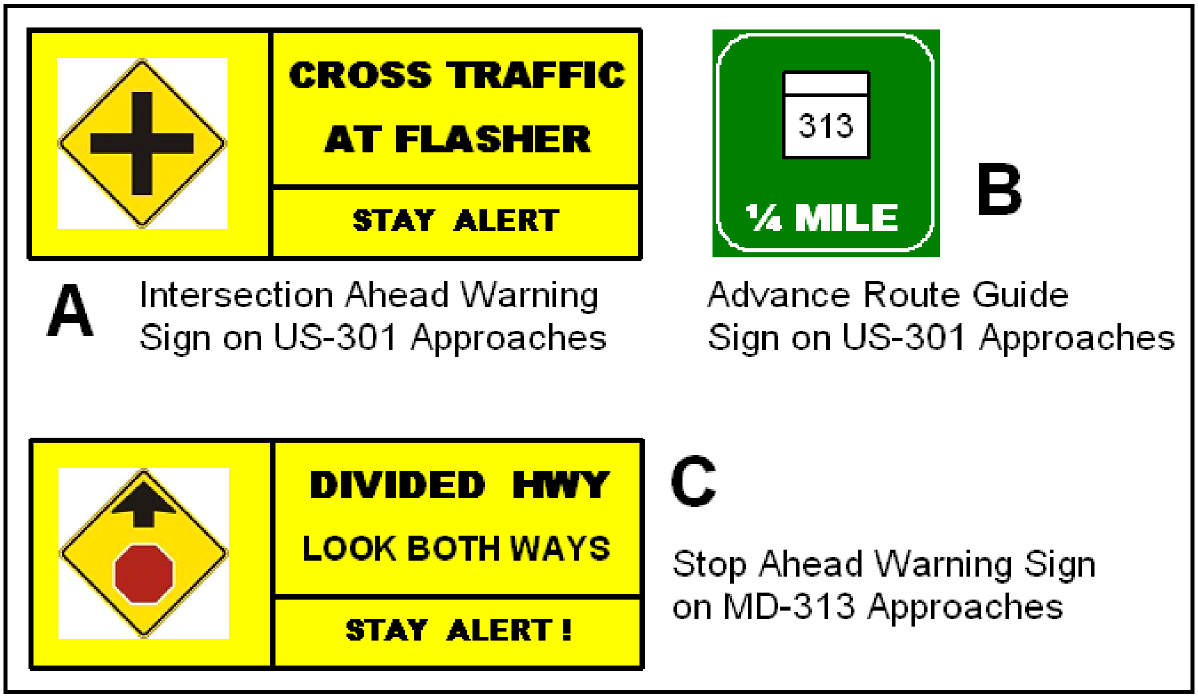


FIGURE 42. US-301/MD-313 signs prior to J-turn intersection conversion

As described previously, the J-turn intersection design implemented at US-301 and MD-313 did not change any of the allowed maneuvers for drivers on US-301; however, a raised directional median, similar to the one shown in the bottom portion of Figure 38, was constructed which separates and offsets the opposing mainline left-turn paths while preventing drivers stopped on the MD-313 approaches from directly crossing or turning left through the median (maneuvers linked to 85 percent of the collisions at the intersection). The curbs are mountable, however, which still allows emergency vehicles to cross directly through the median if they wish. Therefore, all drivers on the MD-313 approaches are forced to turn right with indirect left-turn and crossing maneuvers accommodated via median U-turns located approximately 1,500 feet from the main intersection as shown in Figure 40. Furthermore, the J-turn intersection design at US-301 and MD-313 incorporates U-turn acceleration lanes for passenger cars to utilize and provides loons to accommodate the U-turning path of a WB-50 design vehicle. In addition, several signs were erected to help drivers navigate through the J-turn intersection.

Since maneuvers on US-301 were not affected by the changes, no additional signs were deployed to aid these drivers. The warning sign shown in Figure 42A was left in place, but the freeway style advance route guide sign shown in Figure 42B was replaced with the updated sign illustrated in Figure 43A. For drivers on the MD-313 approaches, a series of new signs were deployed to provide directional guidance, especially to aid drivers making the indirect maneuvers. The signs illustrated in Figures 43B through 43F represent what drivers on the westbound MD-313 approach would see as they approach US-301 and turn north. Drivers on the eastbound MD-313 approach turning south would essentially see a mirror image of these signs. The first sign encountered on the MD-313 approaches is an overhead mounted STOP AHEAD warning sign with additional text indicating the right-turn only condition ahead as shown in Figure 43B. This sign replaced the pre-existing STOP AHEAD sign shown in Figure 42C. Next, the divided highway sign mounted below the STOP sign at the intersection was replaced with a RIGHT TURN ONLY sign as shown in Figure 43C. In addition, the route sign assemblies at the intersection were changed to indicate that a driver needs to turn right in order to reach both directions of US-301 as shown in Figure 43D. After turning right, drivers see the advance guide sign shown in Figure 43E indicating a U-turn is available ahead. This sign is located approximately 300 feet downstream from the main intersection and nearly 1,200 feet in advance of the median U-turn. Finally, the route assembly illustrated in Figure 43F was erected at the median U-turn crossover.

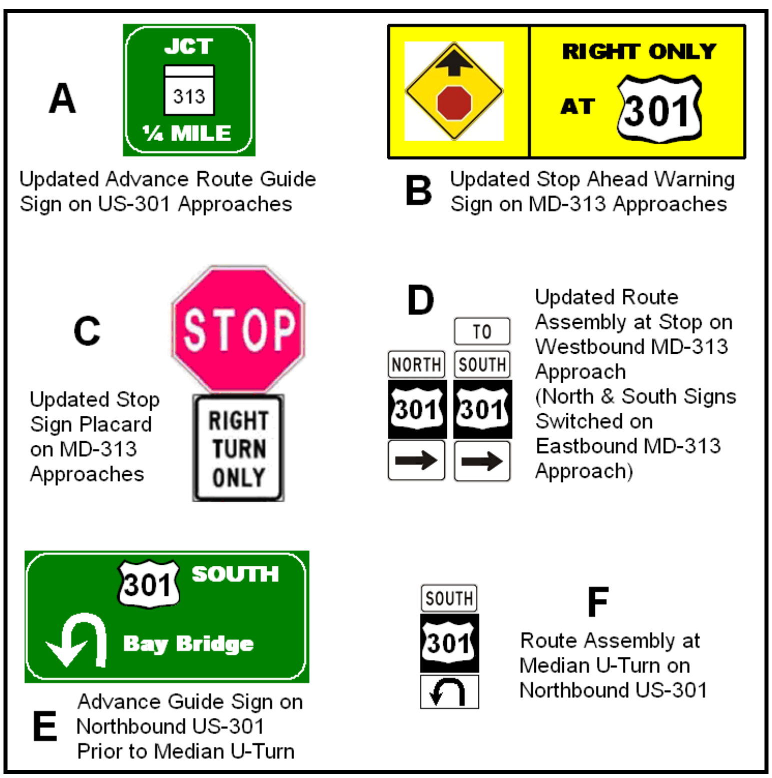


FIGURE 43. US-301/MD-313 signs after J-turn intersection conversion

After construction of the J-turn intersection, crash data were unable to be obtained from the MSHA for the U-turn median crossovers and the weaving areas; however, there were no reported vehicle crashes within a 250 foot radius of the main intersection in four of the next six years. Overall, between 2001 and 2006, there were a total of 4 crashes at the main intersection (0.67 crashes per year), all of which involved property damage only, giving an overall average crash rate of approximately 0.14 crashes per mev. Therefore, there was a 92 percent reduction in annual crash frequency and crash rate per mev after the J-turn intersection was constructed. A collision diagram for the main intersection during the six year after period is shown in Figure 44. An examination of crash types reveals that there were no right-angle collisions at the main intersection in the after period (a 100 percent reduction) and three of the four crashes were single-vehicle collisions (1 overturn, 1 fixed object, and 1 alcohol related). A before-after crash data comparison is shown in Table 11. Even though the expected annual crash frequency increased by 14 percent in the after period based on volume levels, crashes were reduced in all categories.

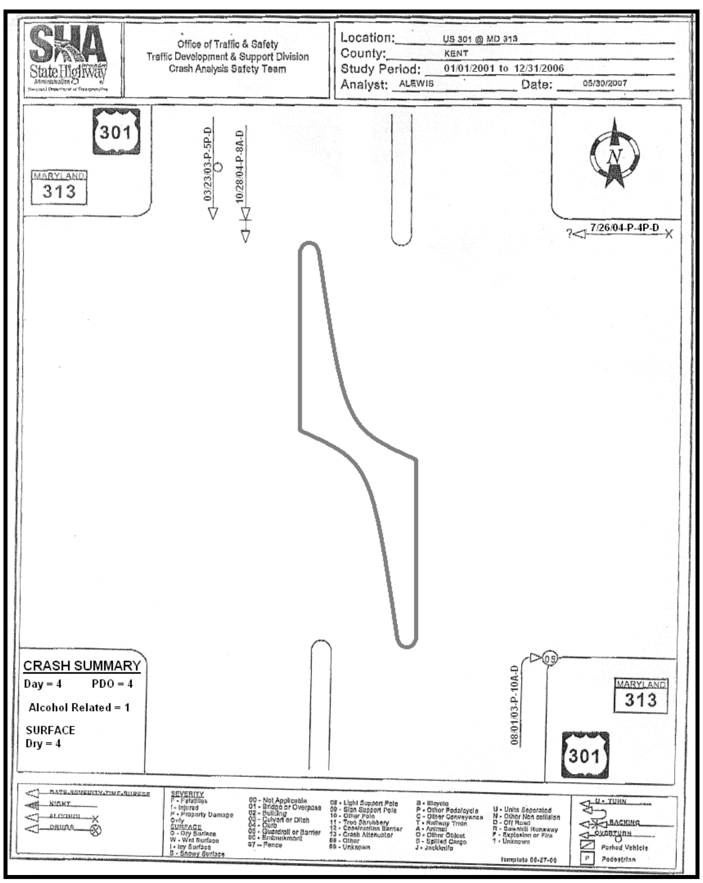


FIGURE 44. Collision diagram at US-301 & MD-313 after J-turn construction (57)

TABLE 11. J-Turn Intersection Before-After Crash Data (US-301 & MD-313)

	BEFORE	AFTER	% CHANGE	SIGNIFICANT DIFFERENCE AT
ESTIMATED ENTERING AADT (US-301) *	10,670	11,240	+ 5.34	
ESTIMATED ENTERING AADT (MD-313) *	1450	1700	+ 17.24	
ESTIMATED TOTAL ENTERING AADT	12,120	12,940	+ 6.77	
EXPECTED CRASH FREQUENCY/YEAR**	3.21	3.66	+ 14.28	
YEARS	4	6		
TOTAL CRASHES ***	33	4		
Crash Frequency/Year	8.25	0.67	-91.92	α = 0.0227 ****
Crash Rate/mev	1.86	0.14	-92.43	
FATAL CRASHES	1	0		
Crash Frequency/Year	0.25	0	-100	α = 0.1955
Crash Rate/mev	0.06	0	-100	
INJURY CRASHES	22	0		
Crash Frequency/Year	5.50	0	-100	α = 0.0226 ****
Crash Rate/mev	1.24	0	-100	
PDO CRASHES	10	4		
Crash Frequency/Year	2.50	0.67	-73.33	α = 0.0275 ****
Crash Rate/mev	0.57	0.14	-75.02	
RIGHT-ANGLE/BROADSIDE CRASHES	22	0		
Crash Frequency/Year	5.50	0	-100	α = 0.0159 ****
Crash Rate/mev	1.24	0	-100	
Far-Side Right-Angle	18	0		
Crash Frequency/Year	4.50	0	-100	α = 0.0212 ****
Crash Rate/mev	1.02	0	-100	
Near-Side Right-Angle	4	0		
Crash Frequency/Year	1.00	0	-100	α = 0.0459 ****
Crash Rate/mev	0.23	0	-100	
IN MEDIAN CRASHES	6	0		
Crash Frequency/Year	1.50	0	-100	α = 0.0514 ****
Crash Rate/mev	0.34	0	-100	
SINGLE-VEHICLE CRASHES	4	3		
Crash Frequency/Year	1.00	0.50	-50.00	α = 0.2448
Crash Rate/mev	0.23	0.11	-53.17	
REAR-END CRASHES	1	1		
Crash Frequency/Year	0.25	0.17	-33.33	α = 0.3954
Crash Rate/mev	0.06	0.04	-37.56	

* AADT on US-301 & MD-313 are averaged from *Maryland's Traffic Volume Maps by County* (*56*). The before period averages 1997-2000 annual values and the after period averages 2001-2006 annual values.

** Maze et al. (*2*) SPF in Table 3 was used to compute these expected values.

*** Total Crashes include crashes within 250 feet of main intersection and do not include collisions at U-turn locations in the after period. That crash data was not available.

**** Significant difference in sample means assuming unequal variances at a 90% level of confidence using a one-tailed t-test.

Because there were more than three years of before and after crash data at this site, statistical comparison of the before and after mean annual crash frequencies shown in Table 11 was performed. To simplify the analysis, the before period was extended to include November and December of 2000 (1/1/1997 through 12/31/2000) even though the J-turn intersection was installed during November of 2000 (there were no crashes during these two months). The six year after period used was 1/1/2001 through 12/31/2006. Using a one-tailed t-test for differences in sample means assuming unequal variances and a 0.10 level of significance ($\alpha = 0.10$), the mean annual crash frequency in the before period was significantly larger for total, injury, property damage only (PDO), right-angle, far-side right-angle, near-side right-angle, and in median crashes.

Originally, local elected officials in the nearby town of Galena, Maryland were opposed to the J-turn intersection configuration; however, after seeing how successful the project has been, they are now very supportive of this intersection design strategy. Due to the overwhelming success of the J-turn intersection at US-301 and MD-313, the MSHA has constructed several more on rural expressways across the state and are planning to design more in the future.

Through their experience and observations, the MSHA has found that passenger cars tend not to use the median U-turn acceleration lanes when mainline volumes are lower (less than 17,000 vpd) and instead U-turn directly into the inside (passing) expressway lane. As a result, the MSHA has not been constructing U-turn acceleration lanes when mainline volumes are in this range and have instead been widening and thickening shoulders (constructing loons) to accommodate WB-67 turning paths. Furthermore, the length the MSHA now uses for the U-turn separation distance depends on mainline traffic volumes, percent trucks, terrain, roadway curvature, and the spacing of existing crossovers in the immediate vicinity. They have observed that 1,500 foot spacing works well at a posted speed limit of 55 mph when mainline volumes are below 20,000 vpd. However, at locations with larger mainline volumes, the MSHA has used separation distances of up to 2,500 feet due to the difficulty of finding gaps when making the required weaving maneuvers. Additionally, the MSHA recommends offsetting the opposing mainline left-turn lanes at the main intersection as much as possible via the directional median to reduce the sight-distance obstruction opposing left-turn vehicles create for each other (see the "Offset Left-Turn Lanes Case Study" presented later in this chapter for more details on this issue). Finally, the MSHA has discovered an approximate volume at which the J-turn intersection configuration operationally begins to break down. The J-turn intersection at US-15 and MD-355 (Hayward Road) on the north outskirts of Frederick, Maryland currently seems to be at or near this breakdown volume. The 2006 ADT on the mainline (US-15) both north and south of the J-turn intersection was approximately 44,000 vpd with approximately 2,150 vpd on MD-355 (*56*). Crash data for this intersection was not obtained; however it is clearly starting to fail operationally with large queues during peak hours. As a result, the MSHA is planning to close this J-turn intersection and replace it with an interchange just upstream.

3.2.3.2 North Carolina Experience

The NCDOT has a Safety Evaluation Group within their Traffic Safety Systems Management Section, the purpose of which is to conduct evaluations of completed safety projects and programs to determine their relative effectiveness in reducing the frequency and severity of motor vehicle crashes (*58*). Several J-turn intersections have been constructed in North Carolina at high-speed TWSC expressway intersections, although the design there is typically referred to as a Superstreet Intersection or a Directional Crossover. Since the construction of these J-turn intersections, a few simple before-after spot safety evaluations have been completed by the NCDOT Safety Evaluation Group. These evaluations were at the intersections of: 1) US-23/74 (Great Smokey Mountain Expressway) and SR-1527/1449 (Steeple Road/Beta Circle Drive), 2) US-64 Business (Knightdale Boulevard) and SR-2234/2500 (Mark's Creek Road), and 3) US-321 (Hickory Boulevard) and SR-1796 (Victoria Court/Clover Drive). Each evaluation is briefly summarized here; however, further details, such as site photos and additional circumstances surrounding each implementation, can be found in the original reports (*59, 60, 61*). The before and after crash data given in these reports are compared in terms of percent change; however, no analyses were conducted in the original reports to determine if the changes were statistically significant. Therefore, additional statistical comparisons were conducted here.

At the intersection of US-23/74 and SR-1527/1449, US-23/74 is a four-lane divided expressway with a posted speed limit of 55 mph. This intersection is located in the middle of a reverse horizontal curve on the mainline. Prior to conversion to a J-turn intersection, it was a traditional TWSC expressway intersection with conventional left-turn lanes on the mainline. The intersection met traffic signal warrants; however, the NCDOT felt that a J-turn intersection design would better preserve the capacity and free-flow integrity of the expressway. As a result, the J-turn intersection conversion was completed on December 28, 1998. An aerial photo of this J-turn intersection is shown in Figure 45 and the conversion involved: 1) construction of a raised directional median with tubular delineators preventing through and left-turn movements from the minor road approaches and separating the mainline left-turn lanes, 2) construction of raised right-turn channelization on both minor road approaches (the channelization island on the north side includes a bulb-out allowing U-turns to be made directly at the intersection by mainline drivers; however no acceleration lane is provided for this movement), 3) conversion from stop control to yield control on both minor road approaches, 4) creation of a much larger turning radius on the southbound minor road approach essentially creating a yield-controlled on-ramp, 5) construction of a U-turn on the Exit 85 ramp approximately ½ mile to the west of the intersection, and 6) posting of U-TURN TRAFFIC ENTERING warning signs on both US-23/74 approaches as well as other navigational guide signs. U-turns to the east are made at a previously existing intersection (US-23/74 and SR-1788 (Hidden Valley Road)) located approximately 1,200 feet downstream.

FIGURE 45. Aerial photo of J-turn intersection at US-23/74 & SR-1527/1449

Before and after collision diagrams for the main intersection are shown in Figure 46 and Table 12 summarizes and compares the before-after crash data for the J-turn intersection conversion at US-23/74 and SR-1527/1449. Overall, there was a 53 percent reduction in total crashes with a 100 percent reduction in right-angle collisions after the J-turn intersection was completed. However, collisions at the downstream U-turn locations increased by 67 percent, although all of these collisions may not have been U-turn related. Because there were more than three years of before and after crash data at US-23/74 and SR-1527/1449, statistical comparison of the before and after mean annual crash frequencies was performed. Using a one-tailed t-test for detecting differences in sample means assuming unequal variances and a 90 percent level of confidence ($\alpha = 0.10$), the mean annual crash frequency in the before period was significantly reduced for total, injury, right-angle, far-side right-angle, and near-side right-angle collisions as shown in Table 12.

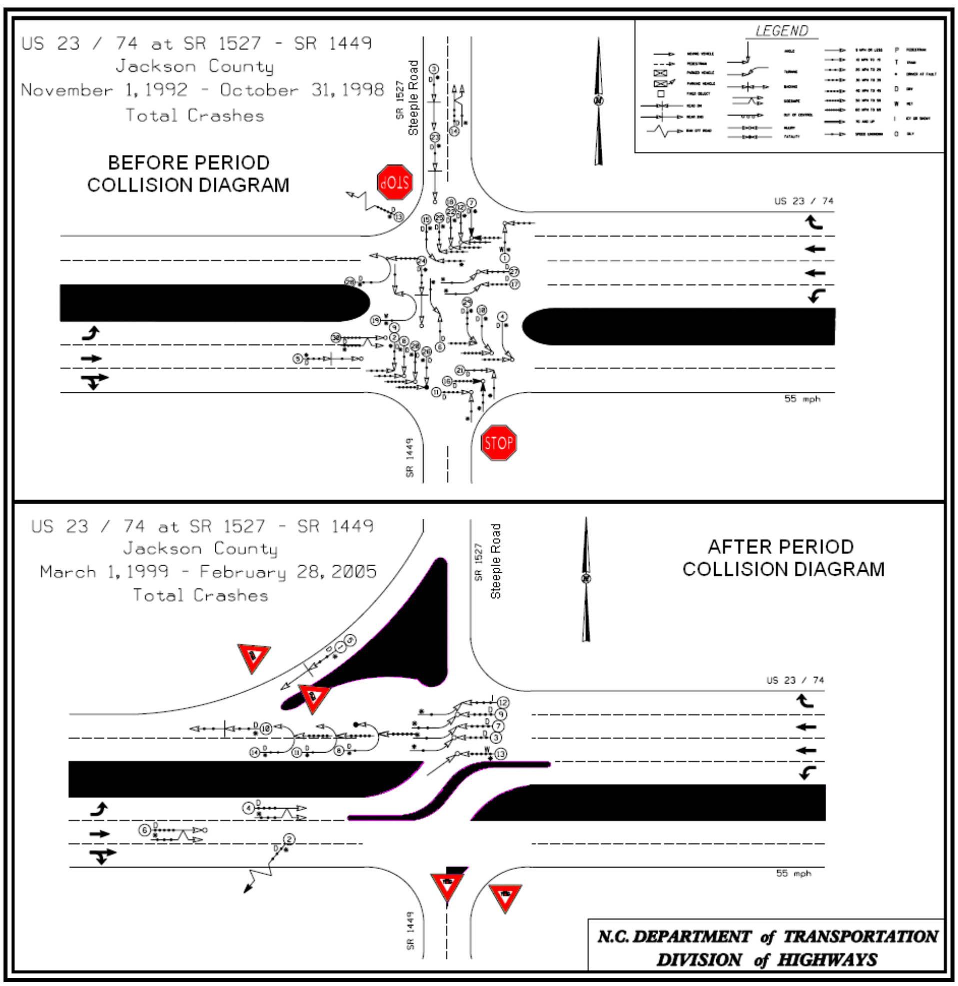


FIGURE 46. Before and after collision diagrams at US-23/74 & SR-1527/1449 (59)

TABLE 12. J-Turn Before-After Crash Data Comparison (US-23/74 & SR-1527/1449)

	BEFORE	AFTER	% CHANGE	SIGNIFICANT DIFFERENCE AT
ESTIMATED TOTAL ENTERING AADT	16,900	20,000	+ 18.34	
YEARS	6	6		
TOTAL CRASHES *	30	14	-53.33	
Crash Frequency/Year	5.00	2.33	-53.33	α = 0.0169 **
Crash Rate/mev	0.81	0.32	-60.57	
FATAL CRASHES	1	1	0	
Crash Frequency/Year	0.17	0.17	0	α = 0.5000
Crash Rate/mev	0.03	0.02	-15.50	
INJURY CRASHES	17	4	-76.47	
Crash Frequency/Year	2.83	0.67	-76.47	α = 0.0099 **
Crash Rate/mev	0.46	0.09	-80.12	
PDO CRASHES	12	9	-25.00	
Crash Frequency/Year	2.00	1.50	-25.00	α = 0.1976
Crash Rate/mev	0.32	0.21	-36.63	
RIGHT-ANGLE/BROADSIDE CRASHES	18	0	-100	
Crash Frequency/Year	3.00	0	-100	α = 0.0017 **
Crash Rate/mev	0.49	0	-100	
Far-Side Right-Angle	9	0	-100	
Crash Frequency/Year	1.50	0	-100	α = 0.0223 **
Crash Rate/mev	0.24	0	-100	
Near-Side Right-Angle	9	0	-100	
Crash Frequency/Year	1.50	0	-100	α = 0.0086 **
Crash Rate/mev	0.24	0	-100	
REAR-END CRASHES	4	3	-25.00	
Crash Frequency/Year	0.67	0.50	-25.00	α = 0.3680
Crash Rate/mev	0.11	0.07	-36.63	
LEFT-TURN LEAVING CRASHES	3	5	+ 66.67	
Crash Frequency/Year	0.50	0.83	+ 66.67	α = 0.2444
Crash Rate/mev	0.08	0.11	+ 40.83	
SIDESWIPE (SAME DIR.) CRASHES	2	2	0	
Crash Frequency/Year	0.33	0.33	0	α = 0.5000
Crash Rate/mev	0.05	0.05	-15.50	
U-TURN CRASHES (At Main Intersection)	2	3	+ 50.00	
Crash Frequency/Year	0.33	0.50	+ 50.00	α = 0.3444
Crash Rate/mev	0.05	0.07	+ 26.75	
SINGLE VEHICLE CRASHES	1	1	0	
Crash Frequency/Year	0.17	0.17	0	α = 0.5000
Crash Rate/mev	0.03	0.02	-15.50	
TOTAL CRASHES (At U-Turn Locations)	3	5	+ 66.67	
Crash Frequency/Year	0.50	0.83	+ 66.67	

* Total intersection-related crashes do not include crashes at downstream U-turns.

** Significant difference in sample means assuming unequal variances at a 90% level of confidence using a one-tailed t-test.

The second J-turn intersection safety evaluation conducted by the NCDOT was at the intersection of Business US-64 (Knightdale Boulevard) and SR-2234/2500 (Mark's Creek Road) near Raleigh, NC. US-64 is a four-lane divided expressway with a posted speed limit of 55 mph and Mark's Creek Road is a two-lane undivided roadway with a posted speed limit of 45 mph. Prior to conversion to a J-turn intersection, the intersection was a traditional TWSC expressway intersection with conventional left-turn lanes on the mainline. At this time, vehicles on Mark's Creek Road had problems crossing and turning left safely at the intersection due to insufficient gaps in the US-64 traffic stream. Of the 45,000 vehicles which used the intersection daily, approximately 400 went straight or turned left from the minor roads; however, these vehicles were involved in 12 of the 21 crashes (57 percent) which occurred during the three year before period (10/1/1998 through 9/30/2001). Therefore, more than half of the crashes were being caused by less than one percent of the motorists at the intersection. As a result, the NCDOT felt that the J-turn intersection configuration would reduce these crashes while minimally impacting traffic progression along US-64; thus the J-turn intersection conversion was completed on November 30, 2001.

An aerial photo of this J-turn intersection is shown in Figure 47. U-turns are made at two previously existing crossovers located approximately 1,100 feet to the west and 3,300 feet to the east. The J-turn intersection conversion involved: 1) construction of a raised directional median preventing through and left-turn movements from the minor road approaches and offsetting the mainline left-turn lanes, 2) modification of the raised right-turn channelization on both minor road approaches, and 3) posting of additional navigational guide signs.

Before and after collision diagrams are shown in Figure 48, while Table 13 summarizes the naïve before-after crash data comparison for the J-turn intersection conversion at US-64 and Mark's Creek Road. Overall, there was a 48 percent reduction in total crashes with reduced crash frequency for all severity levels. Right-angle collisions, which made up 57 percent of the crashes in the before period, were reduced by 92 percent with the complete elimination of far-side right-angle crashes. Left-turn collisions with opposing traffic were reduced by 40 percent and total crashes at the downstream U-turn locations were reduced by 9 percent. However, rear-end and single-vehicle collisions both increased after the J-turn intersection was completed. Because there were three years of

FIGURE 47. Aerial photo of J-turn intersection at US-64 & Mark's Creek Road

before and after crash data at US-64 and Mark's Creek Road, statistical comparison of the before and after mean annual crash frequencies was performed. Using a one-tailed t-test for detecting differences in sample means assuming unequal variances and a 90 percent level of confidence ($\alpha = 0.10$), the mean annual crash frequency was significantly reduced in the after period for right-angle and far-side right-angle collisions as shown in Table 13. However, the increase in rear-end collisions was also statistically significant.

The final J-turn intersection safety evaluation conducted by the NCDOT was at the intersection of US-321 (Hickory Boulevard) and SR-1796 (Victoria Court/Clover Drive) just

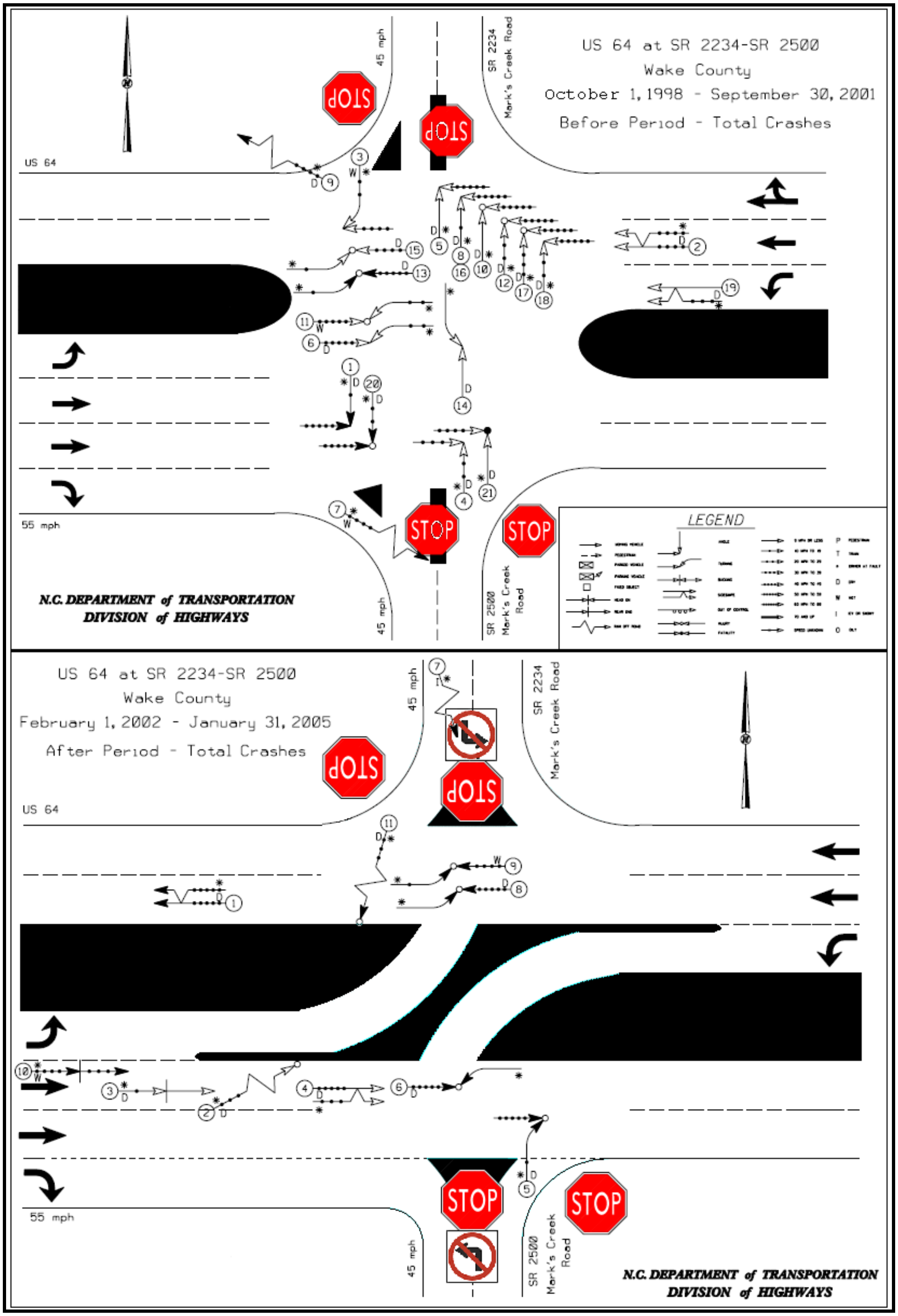


FIGURE 48. Before and after collision diagrams at US-64 & Mark's Creek Road (60)

TABLE 13. J-Turn Before-After Crash Data (US-64 & Mark's Creek Road)

	BEFORE	AFTER	% CHANGE	SIGNIFICANT DIFFERENCE AT
ESTIMATED TOTAL ENTERING AADT	45,000	47,600	+ 5.78	
YEARS	3	3		
TOTAL CRASHES *	21	11	-47.62	
Crash Frequency/Year	7.00	3.67	-47.62	α = 0.1123
Crash Rate/mev	0.43	0.21	-50.48	
FATAL CRASHES	1	0	-100	
Crash Frequency/Year	0.33	0	-100	α = 0.2113
Crash Rate/mev	0.02	0	-100	
INJURY CRASHES	7	6	-14.29	
Crash Frequency/Year	2.33	2.00	-14.29	α = 0.4224
Crash Rate/mev	0.14	0.12	-18.97	
PDO CRASHES	13	5	-61.54	
Crash Frequency/Year	4.33	1.67	-61.54	α = 0.1078
Crash Rate/mev	0.26	0.10	-63.64	
RIGHT-ANGLE/BROADSIDE CRASHES	12	1	-91.67	
Crash Frequency/Year	4.00	0.33	-91.67	α = 0.0368 **
Crash Rate/mev	0.24	0.02	-92.12	
Far-Side Right-Angle	9	0	-100	
Crash Frequency/Year	3.00	0	-100	α = 0.0608 **
Crash Rate/mev	0.18	0	-100	
Near-Side Right-Angle	3	1	-66.67	
Crash Frequency/Year	1.00	0.33	-66.67	α = 0.1955
Crash Rate/mev	0.06	0.02	-68.49	
REAR-END CRASHES	0	2	+Undefined	
Crash Frequency/Year	0	0.67	+Undefined	α = 0.0918 **
Crash Rate/mev	0	0.04	+Undefined	
LEFT-TURN LEAVING CRASHES	5	3	-40.00	
Crash Frequency/Year	1.67	1.00	-40.00	α = 0.2860
Crash Rate/mev	0.10	0.06	-43.28	
SIDESWIPE (SAME DIR.) CRASHES	2	2	0	
Crash Frequency/Year	0.67	0.67	0	α = 0.5000
Crash Rate/mev	0.04	0.04	-5.46	
SINGLE VEHICLE CRASHES	2	3	+ 50.00	
Crash Frequency/Year	0.67	1.00	+ 50.00	α = 0.3623
Crash Rate/mev	0.04	0.06	+ 41.81	
TOTAL CRASHES (At U-Turn Locations)	11	10	-9.09	
Crash Frequency/Year	3.67	3.33	-9.09	

* Total intersection-related crashes do not include crashes at downstream U-turns.

** Significant difference in sample means assuming unequal variances at a 90% level of confidence using a one-tailed t-test.

south of Lenoir, NC. US-321 is a four-lane divided highway with a posted speed limit of 55 mph; however, this intersection is located in a more suburban environment than the previous two examples, with more businesses located along US-321. Prior to conversion to a J-turn intersection, the intersection was a traditional TWSC expressway intersection with conventional right and left-turn lanes on the US-321 approaches. During the three year before period (1/1/1998 to 12/31/2000), the crash experience indicated 10 of the 13 collisions (77 percent) involved motorists attempting to cross or turn left from SR-1796. As a result, the NCDOT felt that a J-turn intersection configuration would reduce the occurrence of these crashes while maintaining traffic progression along US-321 and the conversion was completed on October 13, 2001 at a total cost of $45,000. This J-turn intersection conversion involved: 1) construction of a raised directional median preventing through and left-turn movements from the minor road approaches while offsetting the mainline left-turn lanes, 2) slight modification of the raised right-turn channelization on both minor road approaches, 3) extension of the left-turn storage lanes on the US-321 approaches, and 4) posting of additional navigational guide signs. A quality aerial photo of this intersection was not available; however a location map is shown in Figure 49. U-turns are made at two previously existing intersections located approximately 1,425 feet to the south and 3,220 feet to the north as shown in Figure 49.

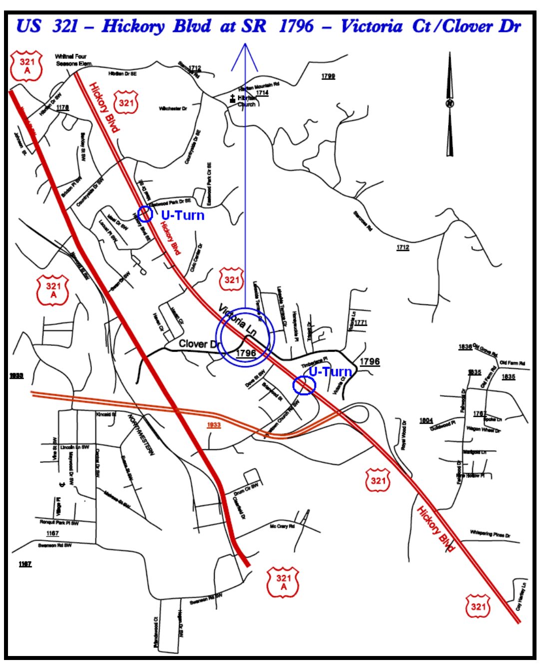


FIGURE 49. Location map for J-turn intersection at US-321 & SR-1796 (61)

Before and after collision diagrams are shown in Figure 50, while Table 14 summarizes the naïve before-after crash data comparison for the J-turn intersection conversion at US-321 and SR-1796. Overall, there was a 69 percent reduction in total crashes with reduced crash severity. Right-angle collisions, which made up 62 percent of the crashes in the before period, were completely eliminated, while total crashes at the U-turn locations were reduced by 64 percent. However, rear-end and left-turn collisions with opposing traffic increased after the J-turn intersection was constructed. Because there were three years of before and after crash data at US-321 and SR-1796, statistical comparison of the before and after mean annual crash frequencies was performed. Using a one-tailed t-test for detecting differences in sample means assuming unequal variances and a 90 percent level of confidence ($\alpha = 0.10$), the mean annual crash frequency was significantly reduced in the after period for total, fatal, PDO, right-angle, and far-side right-angle collisions as shown in Table 14. The increases in rear-end and left-turn collisions with opposing traffic were not statistically significant.

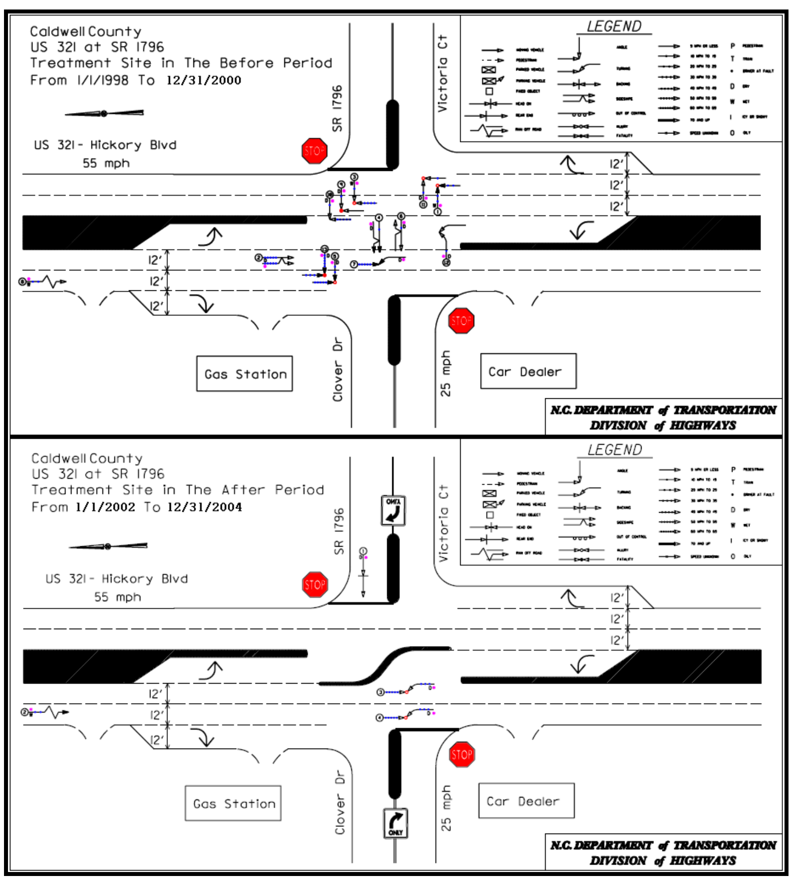


FIGURE 50. Before and after collision diagrams at US-321 & SR-1796 (61)

TABLE 14. J-Turn Before-After Crash Data Comparison (US-321 & SR-1796)

	BEFORE	AFTER	% CHANGE	SIGNIFICANT DIFFERENCE AT
ESTIMATED TOTAL ENTERING AADT	28,600	29,200	+ 2.10	
YEARS	3	3		
TOTAL CRASHES *	13	4	-69.23	
Crash Frequency/Year	4.33	1.33	-69.23	α = 0.0138 **
Crash Rate/mev	0.42	0.13	-69.86	
FATAL CRASHES	2	0	-100	
Crash Frequency/Year	0.67	0	-100	α = 0.0918 **
Crash Rate/mev	0.06	0	-100	
INJURY CRASHES	4	2	-50.00	
Crash Frequency/Year	1.33	0.67	-50.00	α = 0.2185
Crash Rate/mev	0.13	0.06	-51.03	
PDO CRASHES	7	2	-71.43	
Crash Frequency/Year	2.33	0.67	-71.43	α = 0.0557 **
Crash Rate/mev	0.22	0.06	-72.02	
RIGHT-ANGLE/BROADSIDE CRASHES	8	0	-100	
Crash Frequency/Year	2.67	0	-100	α = 0.0286 **
Crash Rate/mev	0.26	0	-100	
Far-Side Right-Angle	5	0	-100	
Crash Frequency/Year	1.67	0	-100	α = 0.0648 **
Crash Rate/mev	0.16	0	-100	
Near-Side Right-Angle	3	0	-100	
Crash Frequency/Year	1.00	0	-100	Not Valid (zero variance in before and after periods).
Crash Rate/mev	0.10	0	-100	
REAR-END CRASHES	0	1	+Undefined	
Crash Frequency/Year	0	0.33	+Undefined	α = 0.2113
Crash Rate/mev	0	0.03	+Undefined	
LEFT-TURN LEAVING CRASHES	1	2	+ 100	
Crash Frequency/Year	0.33	0.67	+ 100	α = 0.3425
Crash Rate/mev	0.03	0.06	+ 95.89	
SIDESWIPE CRASHES	3	0	-100	
Crash Frequency/Year	1.00	0	-100	α = 0.1127
Crash Rate/mev	0.10	0	-100	
SINGLE VEHICLE CRASHES	1	1	0	
Crash Frequency/Year	0.33	0.33	0	α = 0.5000
Crash Rate/mev	0.03	0.03	-2.05	
TOTAL CRASHES (U-Turn Locations)	22	8	-63.64	
Crash Frequency/Year	7.33	2.67	-63.64	

* Total intersection-related crashes do not include crashes at downstream U-turns.
** Significant difference in sample means assuming unequal variances at a 90% level of confidence using a one-tailed t-test.

3.2.4 J-Turn Intersection Summary

The assumed safety benefit of J-turn intersections is that they reduce the potential for right-angle collisions (particularly far-side right-angle collisions) by eliminating direct crossing and left-turn maneuvers from the minor roads at TWSC expressway intersections. Minor road traffic wishing to cross or turn left directly at the intersection are forced to turn right, make a downstream U-turn, and return back to the intersection to complete their desired maneuver. This conflict point management strategy thereby eliminates 20 crossing path conflict points present at a typical TWSC rural expressway intersection and replaces them with less risky conflict points associated with right-turns, U-turns, and weaving maneuvers. Furthermore, by limiting median traffic at the main intersection to only left-turns leaving the mainline, crashes occurring within the median area are expected to be reduced. Finally, by physically separating and offsetting opposing left-turn lanes on the mainline, J-turn intersections may also help reduce collisions between opposing left-turn vehicles and other “left-turn leaving” crashes between left-turn vehicles leaving the expressway and opposing through traffic. As a result, TWSC rural expressway intersections most likely to benefit from J-turn intersection conversion include: 1) intersections with a history of far-side right-angle collisions, collisions within the median, and/or “left-turn leaving” collisions, 2) intersections with high volumes of traffic on the mainline creating infrequent safe gaps for direct crossing or left-turn maneuvers, while still having frequent enough gaps for safe right-turn entry, and 3) intersections with relatively low volumes of traffic crossing or turning left from the minor roads. However, J-turn intersection conversion may potentially lead to an increase in rear-end and sideswipe collisions related to the weaving maneuvers and U-turns.

Limited experience with the J-turn intersection design on rural expressways in Maryland and North Carolina documented in these case studies have shown that this design concept can offer superior safety performance as compared to a typical TWSC rural expressway intersection. Table 15 summarizes the results relating to the target crash types. Given the limited number of sites and the shortcomings of the naïve before-after analysis methodology, definitive conclusions regarding the safety benefits of J-turn intersections cannot be drawn from this study. However, the implementation at the four sites examined completely eliminated far-side right-angle collisions and improved overall safety. The overall safety improvements ranged from 48 to 92 percent. These positive results have led to planned implementations in other states such as Iowa and Minnesota. Future implementation will offer additional opportunities to evaluate the safety benefits and scientifically determine crash reduction factors related to the J-turn intersection design; however, as more STAs begin to implement this strategy, national design guidance is needed. Therefore, the J-turn intersection design could be included in the AASHTO Green Book (*3*) as a design option for rural expressway intersections along with design details for constructing directional median openings and additional design guidance related to minimum median widths and optimum U-turn spacing. Furthermore, the MUTCD (*31*) could include a typical signing plan for a J-turn intersection.

TABLE 15. J-Turn Intersection Conversion Safety Effectiveness Summary

	LOCATION			
ANNUAL CRASH FREQUENCY	**MARYLAND US-301 & MD-313 % CHANGE**	**N. CAROLINA US-23/74 & SR-1527/1449 % CHANGE**	**N.CAROLINA US-64 & Mark's Creek Rd. % CHANGE**	**N. CAROLINA US-321 & SR-1796 % CHANGE**
**Total Crashes ** **	-91.92 *	-53.33 *	-47.62	-69.23 *
Right-Angle Crashes	-100 *	-100 *	-91.67 *	-100 *
Far-Side Right-Angle Crashes	-100 *	-100 *	-100 *	-100 *
Near-Side Right-Angle Crashes	-100 *	-100 *	-66.67	-100
Left-Turn/Opposing Through Crashes	N/A	+ 66.67	-40.00	+ 100
Rear-End Crashes	-33.33	-25.00	+ Undefined * (+ 0.67 crash/yr)	+ Undefined (+ 0.33 crash/yr)
Total Crashes at Downstream U-turns	No Data	+ 66.67	-9.09	-63.64

* Statistically significant change at 90% confidence level using one-tailed t-test.
** Total crashes do not include crashes at downstream U-turns.

Public acceptance of the J-turn intersection design concept proved to be hard to come by in Maryland prior to implementation. Other states, such as Minnesota, have also found the J-turn intersection design concept to be a tough sell at public hearings as the general public perceives traffic signals or interchanges as the only possible solutions to safety issues at rural expressway intersections. As a result, the IA-DOT is currently working with the Center for Transportation Research and Education (CTRE) at Iowa State University to develop a J-turn intersection marketing campaign as a tool to help change public perception of the concept at public meetings prior to construction on rural expressways.

3.3 OFFSET T-INTERSECTION CASE STUDY

3.3.1 Description

Regardless of signing and signalization, at-grade intersections have the potential for vehicle-vehicle collisions as a result of vehicular conflicts. Conflict point management techniques are those treatments which attempt to improve intersection safety by reducing, relocating, or controlling the number and/or type of vehicular conflicts that can occur at an intersection. The key to the effectiveness of these treatments however, is in eliminating the high-risk conflict points. The conflict points with the greatest crash risk (i.e., those accounting for the largest proportion of crashes) at a typical TWSC rural expressway intersection are generally those associated with minor road left-turn and crossing maneuvers (*4*). Therefore, elimination or minimization of these conflict points can be an effective means of improving safety at rural expressway intersections. A second intersection design which accomplishes this objective is the "Offset T-Intersection" illustrated in Figure 51.

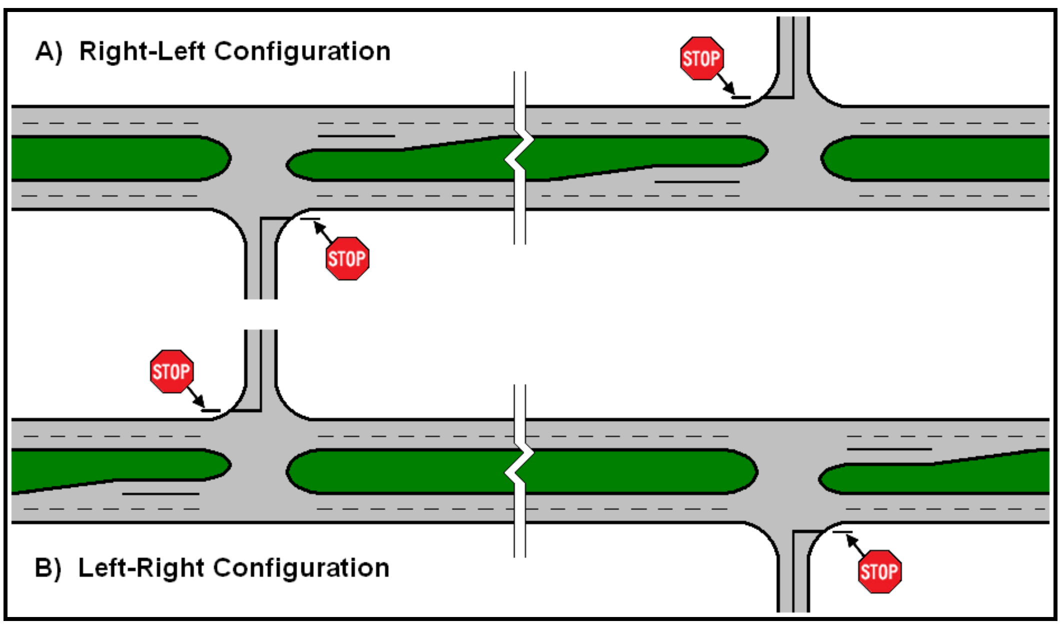


FIGURE 51. Offset T-intersection conceptual schematics

An offset T-intersection is created by separating/staggering two opposing minor road approaches by an appreciable distance along the expressway, thereby creating two three-legged (T) intersections which operate independently while still allowing indirect crossing maneuvers to be made by through traffic on the minor road. The offset T-intersection shown in Figure 51A is described more specifically as a "right-left" (R-L) configuration because the indirect crossing maneuver from the minor road involves a right-turn onto the expressway followed by a left-turn exit (*62*). Conversely, a “left-right” (L-R) configuration (illustrated in Figure 51B) is created when the T-intersections are flip-flopped and the minor road indirect through movement requires left-turn entry onto the expressway followed by a right-turn exit. Theoretically, on expressways, the R-L configuration is preferred over the L-R in terms of both safety and operations because, by requiring the indirect crossing maneuver to be made with right-turn entry, the R-L configuration reduces the number of high-risk left-turn maneuvers from the minor road approaches which are also associated with longer delays.

It has long been acknowledged that three-legged intersections operate more safely than comparable four-legged intersections. Crash models developed by Harwood et al. (*9*) in 1995 revealed that crash frequency and rates at rural, three-legged, unsignalized, divided highway intersections in California are substantially lower than at their four-legged counterparts. Furthermore, Bared and Kaisar (*62*) found that collisions at rural stop-controlled T-intersections of four-lane and two-lane roadways are less severe than collisions at similar four-legged intersections (see Table 16). The reasons for this are easy to understand. Three-legged intersections are less complex, lead to less driver confusion, and have almost 75 percent fewer conflict points at which conflicting traffic streams cross, merge, or diverge. A typical three-legged expressway intersection has only 11 total conflict points (see Figure 11) as compared with 42 at a typical four-legged expressway intersection (see Figure 2). However, more importantly, three-legged intersections minimize the maneuvers and the associated conflict points that have been observed to be over-represented in rural expressway intersection crashes (all the far-side conflict points associated with minor road crossing maneuvers and all but one of the far-side conflict points associated with minor road left-turns are eliminated). In addition, the number of conflict points within the median crossover is dramatically reduced.

TABLE 16. Severity Comparison of Rural 4x2-Lane Stop-Controlled Intersections (*62*)

	California Data for 4x2-Lane Intersections	
CRASH SEVERITY	**FOUR-LEGGED INTERSECTIONS (% of Crashes)**	**T-INTERSECTIONS (% of Crashes)**
FATAL	2	1
INJURY	59	51
PDO	39	48

When two T-intersections combine to form an offset T-intersection configuration, the total number of conflict points is 26 (11 conflict points at each T-intersection plus 2 merge and 2 diverge points in between), regardless of whether it is a R-L or a L-R configuration. A conflict point diagram for a R-L offset T-intersection configuration is illustrated in Figure 52. Therefore, converting a four-legged TWSC expressway intersection into an offset T-intersection reduces the total number of conflict points by 38 percent and would be expected to reduce far-side right-angle collisions. Bared and Kaisar (*62*), estimated that this type of conversion would reduce total crashes between 40 and 60 percent where expressway design speeds are greater than 50 mph and the total entering traffic volumes are less than 25,000 vpd. NCHRP 500, Volume 5 (*16*) lists this strategy as tried, meaning that its safety effectiveness has not yet been determined and, although no volume thresholds are given, it states that the success of this type of conversion largely depends upon the through volumes emanating from the minor road, with higher volumes leading to excessive turning movements, weaving maneuvers, and delay. Other disadvantages associated with offset T-intersections include increased travel time/distance and potential confusion for drivers making a through movement on the minor road.

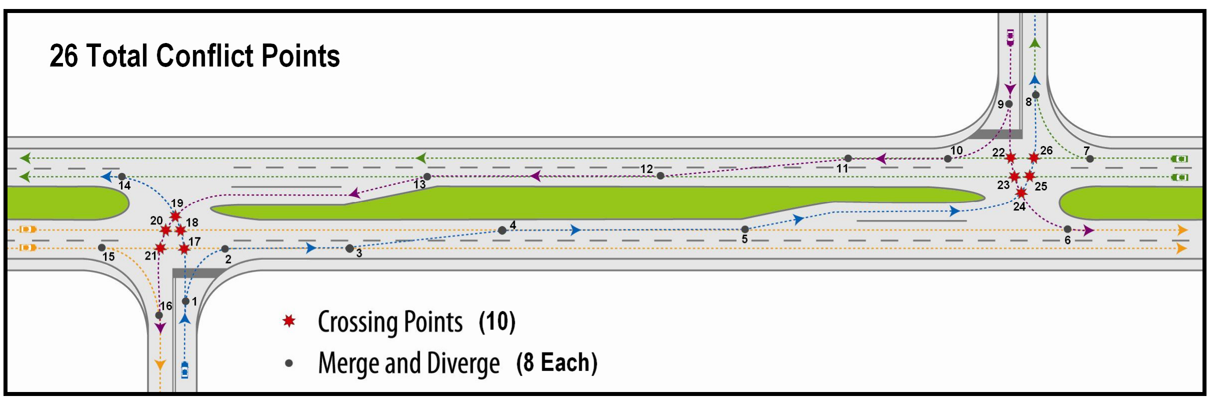


FIGURE 52. Conflict point diagram for an offset T-intersection

3.3.2 Existing Design Guidance

The offset T-intersection design is one possible countermeasure between a typical TWSC rural expressway intersection and an interchange which still allows a reasonable level of accessibility to through drivers on the minor road at a much lower price tag. However, national design guidance for this type of intersection generally does not exist. In fact, this type of intersection is only briefly mentioned in Chapter 9 of the AASHTO Green Book (*3*).

On page 581, offset T-intersections are discussed as a possible method for realigning acute angle intersections; however, this is the only context in which they are mentioned in the entire policy. Green Book Exhibits 9-18C and 9-18D (Figure 53) are provided within the Green Book to illustrate the concept of R-L and L-R realignment configurations, respectively.

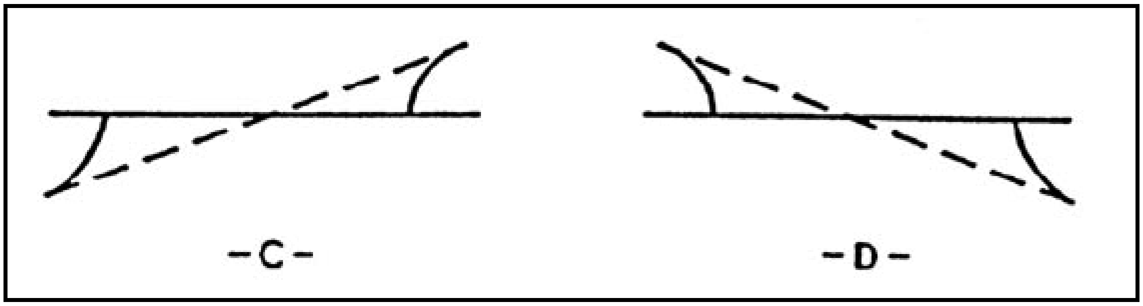


FIGURE 53. Green Book Exhibit 9-18; realignment variations at intersections (3)

When discussing offset T-intersections on page 581, the Green Book states:

> *Realignment of the minor road, as shown in Exhibit 9-18C, provides poor access continuity because a crossing vehicle must reenter the minor road by making a left turn off the major highway. This design arrangement should only be used where traffic on the minor road is moderate, the anticipated minor road destinations are local, and the through traffic on the minor road is low. Where the alignment of the minor road is as shown in Exhibit 9-18D, access continuity is better because a crossing vehicle first turns left onto the major road (e.g., a maneuver that can be done by waiting for an opening in the through traffic stream) and then turns right to reenter the minor road, thus interfering little with through traffic on the major road. Where a large portion of the traffic from the minor road turns onto the major road rather than continuing across the major road, the offset-intersection design may be advantageous regardless of the right or left entry.*

These comments regarding R-L versus L-R configurations may be true where the major road is a two-lane highway; however, it is believed that a R-L configuration would be preferred in terms of both safety and operations where the major road is a rural expressway due to the reasons mentioned previously in the "Description" section. No research was found which examines this issue on expressways; however, Mahalel et al. (*63*) examined the R-L versus L-R issue for offset T-intersections on rural two-lane highways and reported that L-R configurations had greater reductions in injury crashes, but R-L configurations created less delay and had higher capacity.

A second design issue related to offset T-intersections is the spacing required between them. Ideally, the two T-intersections should be spaced far enough apart so that they will each operate independently, allowing a through vehicle on the minor road adequate space to merge across the expressway lanes and safely enter the opposite minor roadway without causing undue interference to through expressway traffic. However, similar to the J-turn intersection, selection of the most appropriate separation/offset distance is a trade-off between providing sufficient space for safe/functional weaving areas and adequate left/right-turn storage, while minimizing the travel distance/time of the indirect crossing maneuver from the minor road. No national design guidance is provided regarding the minimum offset distance and the safety impacts of the separation distance are still unclear; however, some research has been conducted in this area. For 65 mph divided four-lane roads, Bared and Kaisar (*62*) suggest that interference to expressway traffic is minimized when the T-intersections are offset by 141 feet for a R-L configuration and by 235 feet for a L-R configuration. However, these distances seem extremely short as the minimum spacing between median openings currently used by STAs in rural areas ranges from 500 feet to a half mile, with an average minimum spacing of 1,400 feet (*28*). Therefore, these distances would seem more appropriate for offset spacing under high-speed conditions, although more research needs to be conducted to determine the optimum spacing for offset T-intersections located on rural expressways.

Another design issue related to offset T-intersections is the design of the T-intersections themselves. There are three different T-intersection designs that could potentially be used: a typical T, a channelized T, or a continuous flow T. These three T-intersection designs are illustrated in Figure 54. Further research is necessary to determine which of these designs performs best in terms of safety and operations; however, the continuous flow T-intersection was developed specifically for T-intersections in which a minor collector roadway ends at a major highway (*64*). The continuous flow T-intersection has been more commonly referred to as a continuous green T-intersection when it is signalized. Hummer and Boone (*65, 66*) have previously addressed some of the advantages and disadvantages associated with a signalized continuous green T-intersection in relation to its use on urban and suburban arterials. However, no research was found on the unsignalized application of this configuration in rural settings, which is the intended treatment here.

Finally, the biggest issue with converting a four-legged intersection into an offset T configuration is acquiring the necessary right-of-way to allow for the relocation/realignment of one of the minor roadway legs, especially if the land along the existing right-of-way is already in use. However, in rural areas, this may not be as much of an issue and frontage roads could be constructed to connect the old minor roadway approach to its new intersecting location. Because retrofitting could prove to be difficult, identifying opportunities to create offset T-intersections should be considered as an extremely important aspect of the initial expressway corridor development process.

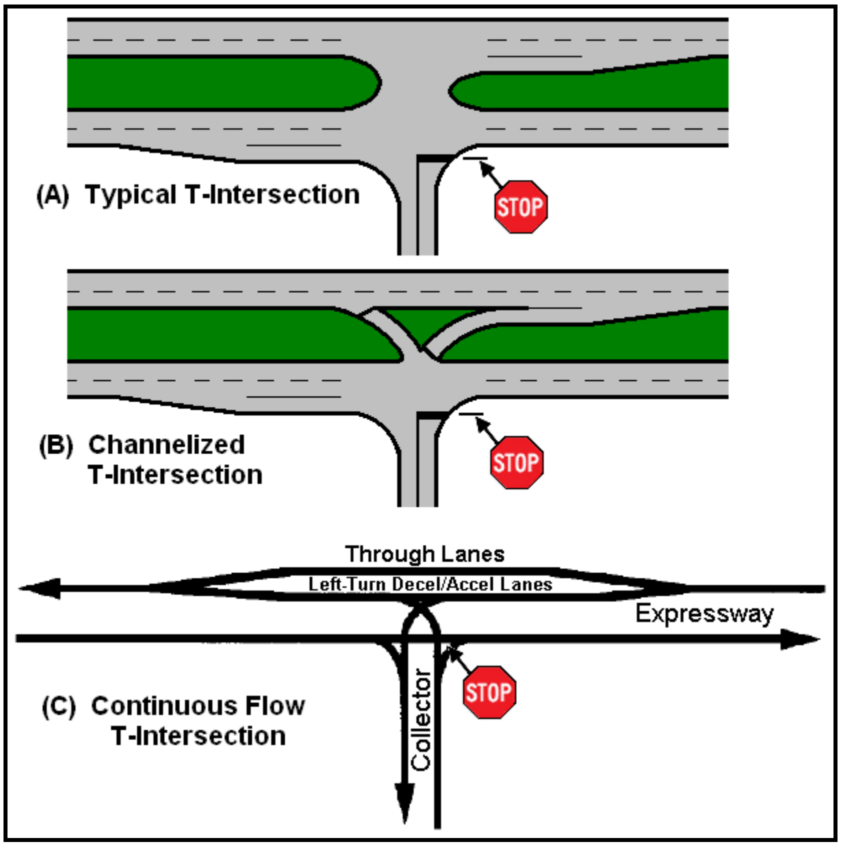


FIGURE 54. Types of T-intersections

3.3.3 Case Studies of Implementation & Safety Effectiveness

Finding examples of offset T-intersections on rural expressways proved to be very challenging for this project, not to mention finding four-leg to offset T-intersection conversion projects where before and after crash data could be obtained and the safety effectiveness examined. However, for this case study, the experiences of NDOR, the Oregon Department of Transportation (ODOT), and the City of Fort Dodge, Iowa were explored.

3.3.3.1 Nebraska Experience

One example of an offset T-intersection was found approximately 20 miles south of Lincoln, Nebraska forming the East and West Junctions of US-77 (Homestead Expressway) and Nebraska Highway 41 (N-41). An aerial photo of this L-R offset T-intersection configuration is shown in Figure 55. US-77 is a four-lane divided rural expressway between Beatrice and Lincoln which has a speed limit of 65 mph. The East and West Junctions of US-77 and N-41 are both typical T-intersections which are offset by approximately 1.50 miles; although, this may not be considered a "true" offset T-intersection since there is a four-legged intersection with a gravel county road in between as shown in Figure 55. However, the county road is a very low volume roadway and east/westbound through traffic on N-41 would certainly utilize the offset T-intersection.

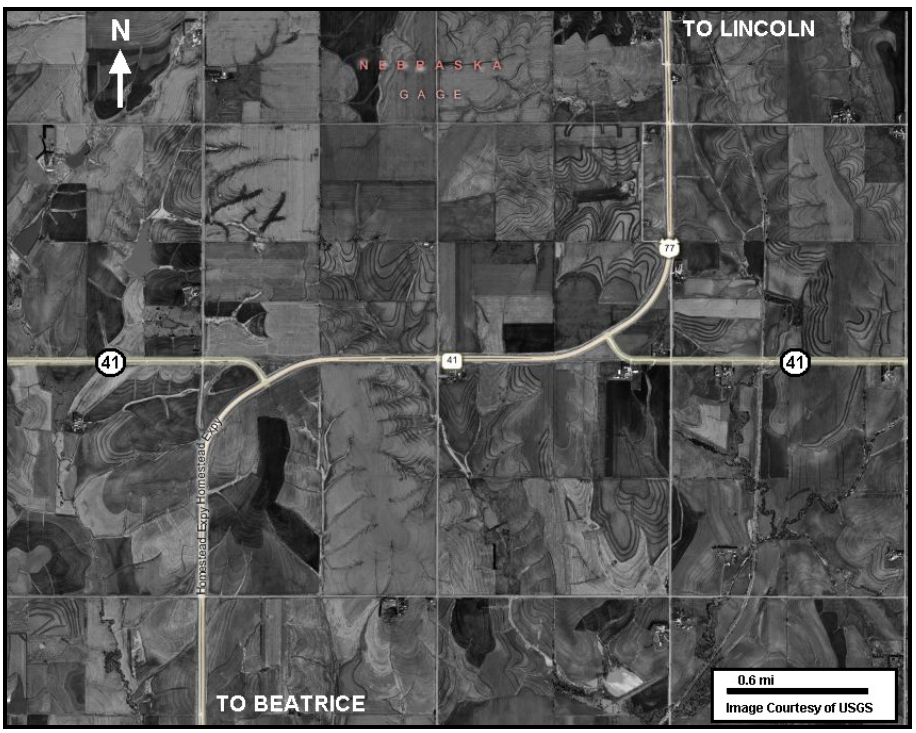


FIGURE 55. Aerial photo of L-R offset T-intersection in Nebraska

The conversion of US-77 from a two-lane undivided highway to expressway standards was completed in April of 1992 for the portion shown in Figure 55. At that time, the offset T-intersection was created; however, the original intent was not to create an offset T-intersection, it just happened to occur during the corridor development process as a result of design convenience and a desire to reduce the skew of the two intersections. Because this was not a direct conversion of a four-legged expressway intersection into an offset T configuration, no before-after crash data exists to examine the effectiveness of a conversion. However, the crash data at these two T-intersections was examined just to get an idea of the crash history at this L-R offset T configuration. Between April of 1992 and December of 2000 (8.75 years), the two T-intersections combined experienced a total of 13 intersection-related crashes (1 fatal, 8 injury, and 4 PDO), equating to 1.49 crashes per year. Due to the fact that both of the T-intersections are located on horizontal curves, the crash experience at this particular L-R offset T configuration is most likely elevated. For comparison purposes, a single four-legged expressway intersection with similar entering volumes would have been expected to average 2.18 crashes per year over this same time frame based on the Maze et al. (*2*) equation given in Table 3.

3.3.3.2 Oregon Experience

A second example of an offset T-intersection was found at the intersection of Oregon Highway 34 (OR-34) and Oakville Road, located approximately 6 miles east of Corvallis, Oregon. An aerial photo of this R-L offset T configuration is shown in Figure 56. In this area, OR-34 is a four-lane divided highway with a narrow flush paved median. Prior to 1995, the intersection was four-legged. Sometime in 1995, ODOT converted the intersection into a R-L offset T by moving the south leg approximately one quarter mile (1320 feet) to the west, which also reduced the skew of the former intersection. Figure 57 illustrates some of the signage used along westbound OR-34 in advance of this offset T configuration.

FIGURE 56. Aerial photo of R-L offset T-intersection in Oregon

FIGURE 57. Offset T-intersection signage at OR-34 & Oakville Road

Before and after crash data for this conversion project were provided by ODOT and are shown in Table 17 (specific crash type information was not obtained). The data indicate that the conversion from a four-legged expressway intersection into a R-L offset T configuration resulted in a 53 percent reduction in total annual crashes with a 72 percent reduction in annual fatal/severe injury crashes. Therefore, the overall crash reduction at this location is consistent with what was estimated by Bared and Kaisar (*62*).

TABLE 17. Before-After Crash Data for Oregon R-L Offset T-Intersection Conversion

	BEFORE *	AFTER **	% CHANGE	SIGNIFICANT DIFFERENCE AT
YEARS	5	9		
TOTAL CRASHES	19	16		
Crash Frequency/Year	3.80	1.78	-53.22	$\alpha = 0.0742$ ***
FATAL CRASHES	1	0		
Crash Frequency/Year	0.20	0	-100	$\alpha = 0.1870$
SEVERE INJURY CRASHES	7	4		
Crash Frequency/Year	1.40	0.44	-68.25	$\alpha = 0.0707$ ***
LESS SEVERE CRASHES	11	12		
Crash Frequency/Year	2.20	1.33	-39.39	$\alpha = 0.1728$

* Before crash data (1990-1994) is for OR-34 & Oakville Road (4-legged).

** After crash data (1996-2004) is combined for both East & West T-intersections.

*** Significant difference in sample means assuming unequal variances at a 90% level of confidence using a one-tailed t-test.

Because there were more than three years of before and after crash data at this site, statistical comparison of the before and after mean annual crash frequencies given in Table 17 was performed. Using a one-tailed t-test for differences in sample means assuming unequal variances and a 90 percent level of confidence ($\alpha = 0.10$), the mean annual crash frequency was significantly reduced in the after period for total and severe injury crashes.

3.3.3.3 Fort Dodge, Iowa Experience

The final example of an offset T-intersection was found in a suburban location on the west side of Fort Dodge, Iowa. The intersection of US-169 (a four-lane divided highway) and Avenue G was converted from a four-legged intersection into a L-R offset T-intersection in November of 2002. Before and after aerial photos of the intersection are shown in Figure 58. A roadway level view of the after condition is shown in Figure 59. The offset distance between the two T-intersections is approximately 1500 feet.

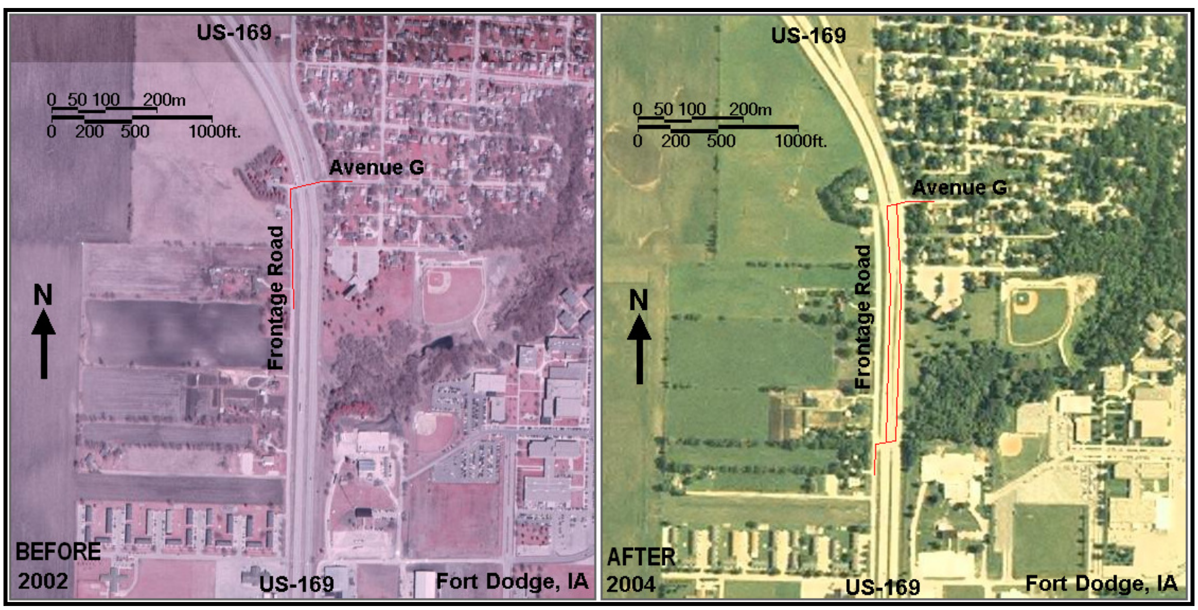


FIGURE 58. Before & after aerial photos of L-R offset T-intersection in Fort Dodge, IA

Before and after crash data for this conversion project were obtained from the Iowa Traffic Safety Data Service (ITSDS) and are presented in Table 18. In the three year before period (1999 – 2001), the four-legged intersection of US-169 and Avenue G averaged 3.33 crashes annually. In the three year after period (2003 – 2005), the two T-intersections combined to average just 2.00 crashes per year. Therefore, the overall crash reduction for this L-R offset T conversion project was 40 percent, which is consistent with what was

FIGURE 59. Looking north from south T-intersection at Fort Dodge offset T-intersection

TABLE 18. Before-After Crash Data; Fort Dodge Offset T-Intersection Conversion

	BEFORE *	AFTER **	% CHANGE	SIGNIFICANT DIFFERENCE AT
YEARS	3	3		
TOTAL CRASHES	10	6		
Crash Frequency/Year	3.33	2.00	-40.00	α = 0.1026
FATAL CRASHES	1	1		
Crash Frequency/Year	0.33	0.33	0	α = 0.5000
INJURY CRASHES	4	3		
Crash Frequency/Year	1.33	1.00	-25.00	α = 0.3709
PDO CRASHES	5	2		
Crash Frequency/Year	1.67	0.67	-60.00	α = 0.2534
RIGHT-ANGLE CRASHES	7	4		
Crash Frequency/Year	2.33	1.33	-42.86	α = 0.1833
Far-Side Right-Angle Crashes	5	0		
Crash Frequency/Year	1.67	0	-100	α = 0.0648 ***
Near-Side Right-Angle Crashes	2	4		
Crash Frequency/Year	0.67	1.33	+ 100	α = 0.1151
OTHER CRASH TYPES	3	2		
Crash Frequency/Year	1.00	0.67	-33.33	α = 0.3257

* Before crash data (1999-2001) is for US-169 & Avenue G (4-legged).

** After crash data (2003-2005) is combined for both North & South T-intersections.

*** Significant difference in sample means assuming unequal variances at a 90% level of confidence using a one-tailed t-test.

estimated by Bared and Kaisar (*62*). In addition, right-angle crashes were reduced by 43 percent, with the targeted crash type, far-side right-angle crashes, completely eliminated. Because there were three years of before and after crash data at this site, statistical comparison of the before and after mean annual crash frequencies was performed. Using a one-tailed t-test for detecting differences in sample means assuming unequal variances and a 90 percent level of confidence ($\alpha = 0.10$), the reduction in far-side right-angle crash frequency was statistically significant; however, the changes in all other crash types were not.

3.3.4 Offset T-Intersection Summary

The assumed safety benefit of offset T-intersections is that they reduce the potential for right-angle collisions (particularly far-side right-angle collisions) by eliminating direct crossing maneuvers from the minor road at TWSC expressway intersections. By staggering the minor road approaches, minor road traffic wishing to cross the expressway are forced to make the maneuver indirectly. If the offset T-intersection is a R-L configuration, the indirect crossing is made via a right-turn onto the expressway followed by a left-turn exit (vice versa for a L-R configuration). As a result, this conflict point management strategy eliminates 16 conflict points present at a typical TWSC rural expressway intersection (14 of which are crossing path conflict points) and spreads the remaining conflict points out over a larger area. Therefore, TWSC rural expressway intersections most likely to benefit from offset T-intersection conversion include those with a pattern of far-side right-angle collisions combined with lower volumes of through traffic on the minor roads. However, offset T-intersection conversion may potentially lead to an increase in rear-end and sideswipe collisions related to the required weaving maneuvers.

Limited experience with the offset T-intersection on expressways in Oregon and Iowa documented in this case study have shown that the design concept can offer superior safety performance as compared to a typical TWSC rural expressway intersection. Table 19 summarizes these results. Given the limited number of sites and the shortcomings of the naïve before-after crash analysis methodology, definitive conclusions regarding the safety benefits of offset T-intersections cannot be drawn from this study. However, the overall crash reduction was in the 40 to 60 percent range estimated by Bared and Kaisar (*62*) and, as expected, a R-L configuration seems to provide additional safety benefits.

TABLE 19. Offset T-Intersection Conversion Safety Effectiveness Summary

	LOCATION	
ANNUAL CRASH FREQUENCY	RURAL OREGON OR-34 & Oakville Rd. (R-L Configuration) (% CHANGE)	SUBURBAN IOWA US-169 & Avenue G (L-R Configuration) (% CHANGE)
Total Crashes	-53.22 *	-40.00
Right-Angle Crashes	No Data	-42.86
Far-Side Right-Angle Crashes	No Data	-100 *
Near-Side Right-Angle Crashes	No Data	+ 100

* Statistically significant change at 90% confidence level using one-tailed t-test.

Future implementation of the offset T-intersection design concept on expressways will offer additional opportunities to evaluate its safety benefits and scientifically determine crash reduction factors; however, as more STAs begin to implement this strategy, national design guidance is needed. The offset T-intersection could be included as a design option for rural expressway intersections in the AASHTO Green Book (*3*) along with general design guidance related to optimum spacing, R-L versus L-R configurations, and the different types of T-intersections that could be used (i.e., typical, channelized, or continuous flow). Further research in these areas is likely required. Furthermore, the section entitled, "Ultimate Development of Four-Lane Divided Arterials" within Chapter 7 of the Green Book (*3*) should mention the importance of identifying opportunities to create offset T-intersections during the initial corridor planning process as rural two-lane undivided highways are being upgraded to divided highways. Finally, a typical signing plan for an offset T-intersection could be incorporated into the MUTCD (*31*).

3.4 JUGHANDLE INTERSECTION CASE STUDY

3.4.1 Description

To motorists, a rural expressway may appear to be a freeway. As such, expressway drivers may have the same expectations of the expressway as they have for a freeway/interstate facility. These expectations include full access control (i.e., no at-grade intersections), free flow (i.e., no traffic signals), exits/entrance ramps on the right, slower traffic keeping right, and relatively low speed differentials between vehicles traveling in the same direction (*27, 67*). However, at-grade intersections on expressways create a setting which conflicts with these driver expectations. While providing exclusive left-turn lanes within the median on expressway intersection approaches removes left-turn leaving vehicles from the high-speed through lanes, reduces speed differentials in those lanes, and improves overall intersection safety and capacity; left-turning vehicles still violate the exit on the right expectation and speed differentials in the passing lane may not be reduced as much as expected since left-turning vehicles exiting the expressway must first merge into the passing lane prior to entering the left-turn deceleration lane.

One rural expressway intersection design alternative which removes left-turning vehicles from the high-speed expressway traffic stream and only allows turns to be made from the right-hand lane is the "Jughandle Intersection". The New Jersey Department of Transportation (NJDOT) Roadway Design Manual (*68*) defines a "jughandle" as, "an at-grade ramp provided at or between intersections to permit motorists to make indirect left-turns and/or U-turns." The jughandle intersection consists of two jughandles (at-grade one-way roadways/ramps) in opposite quadrants of the intersection which diverge to the right and indirectly accommodate all left-turns leaving the expressway. Two versions of the jughandle intersection are shown in Figure 60. Figure 60A illustrates a "near-side" jughandle intersection with diagonal ramps located in advance of the intersection to accommodate all turning traffic (including right-turns) leaving the expressway. Indirect left turns from the expressway are accomplished by exiting at the near-side ramp, turning left onto the cross street at the ramp terminal, and then crossing the expressway as through traffic on the minor road. Figure 60B presents a "far-side" jughandle intersection with loop ramps located beyond the intersection. Indirect left turns from the expressway are accomplished by traveling through the main intersection, exiting to the right at the loop ramp, merging onto the cross street at the ramp terminal, and then crossing the expressway as through traffic on the minor road. With this configuration, right-turning traffic leaving the expressway turns right at the main intersection in a traditional manner.

Jughandle intersections are conflict point management techniques because they eliminate direct left-turns from the expressway and reduce the 42 conflict points associated with TWSC expressway intersections (Figure 2). Conflict point diagrams for both jughandle intersection types are shown in Figure 60. The near-side configuration has a total of 28 conflict points while the far-side configuration has a total of 26.

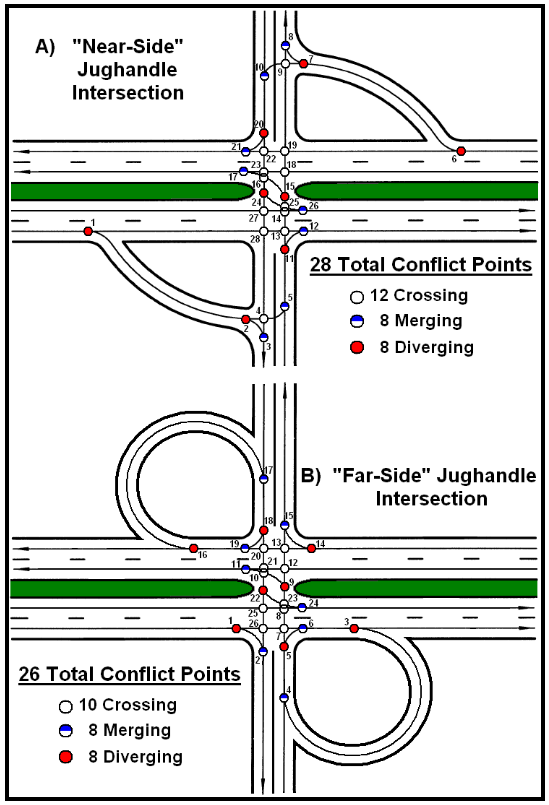


FIGURE 60. Conflict point diagrams for jughandle intersections (adapted from (69))

Both jughandle intersection designs replace a direct left-turn from the expressway at the main intersection with an indirect jughandle maneuver which includes a crossing movement from the minor road. As a result, longitudinal gap selection through head-on, oncoming traffic is replaced with lateral gap selection through side-to-side traffic and more minor road traffic is sent directly through the median. Therefore, based on crash trends at typical TWSC rural expressway intersections, a lower risk direct left-turn movement from the mainline is being replaced by a higher risk crossing maneuver from the minor road (recall that the greatest crash risk movements at a typical TWSC rural expressway intersection are generally those associated with minor road left-turn and crossing maneuvers (*4*)). Therefore, this strategy may increase right-angle collisions. On the other hand, human factors research states that longitudinal gap selection tends to be a more challenging task for drivers because the displacement of a vehicle viewed longitudinally has a smaller visual effect (i.e., subtle changes in vehicle size as viewed against a constant background) than when the same vehicle displacement is viewed laterally (i.e., vehicle moving across a changing background where it passes in front of one fixed reference point after another). In other words, lateral movement results in a higher degree of relative motion and is thus easier to detect visually (*70*). Therefore, the jughandle intersection design may actually simplify the gap selection process for left-turning traffic leaving the expressway.

Jughandle intersections are most appropriate at locations with operational and safety problems resulting from difficulties accommodating left-turn demand (*16*). These problems may occur for a variety of reasons including inadequate median width to provide conventional or offset left-turn lanes, inability to provide adequate left-turn vehicle storage, high left-turning volumes, or left-turn sight distance issues. Therefore, according to a number of sources, jughandle intersections should be considered on arterials with narrow right-of-way, narrow medians, large volumes of through traffic, and low to moderate left-turn volumes on both the arterial and the intersecting roadway (*30, 64, 66, 71*). With jughandle intersections, when left-turn or minor road volumes are proportionately high, the potential for storage problems exists on the minor roads and/or on the jughandle ramps. In addition, jughandle intersections are particularly appropriate at signalized intersections because they can reduce delay by allowing the main intersection to operate under simple two-phase signalization (*30, 35, 69, 71*).

Because jughandle intersections have been predominantly used at signalized intersections, no safety or operational comparisons between traditional TWSC expressway intersections and unsignalized jughandle intersections were uncovered in previous research. NCHRP 500, Volume 5 (*16*) lists this strategy as tried, meaning that its safety effectiveness has not yet been determined, but goes on to say that jughandle intersections are expected to reduce the frequency of rear-end collisions involving left-turning traffic leaving the mainline as well as broadside collisions between left-turning traffic leaving the mainline and opposing through vehicles. However, it is hypothesized here that a jughandle intersection may actually increase the frequency of right-angle collisions due to the increased volume of crossing traffic from the minor road.

Although no safety comparisons were found for unsignalized intersections, Jagannathan et al. (*69*) recently compared the safety of 50 conventional signalized intersections with 44 signalized jughandle intersections in New Jersey and found that jughandles tend to: 1) reduce the severity of intersection crashes, 2) reduce head-on and left-turn crash rates, and 3) increase rear-end crash rates. The study also looked at the safety of near-side versus far-side signalized jughandle configurations and found that the near-side configuration had significantly higher crash rates overall as well as larger right-angle and left-turn crash rates. Furthermore, a separate report by Jagannathan (*72*) analyzed the operational performance of signalized jughandle intersections versus conventional signalized intersections; however, those results are not as applicable here since unsignalized jughandle applications are the treatment under investigation in this thesis.

In addition to providing exits on the right, reducing the number of conflict points, and replacing longitudinal left-turn gap selection with lateral gap selection, the jughandle intersection design offers other advantages and disadvantages. General advantages and disadvantages of the jughandle intersection design are listed in Table 20. More specific advantages and disadvantages associated with the near-side and far-side jughandle configurations are summarized in Tables 21 and 22, respectively.

3.4.2 Existing Design Guidance

The current edition of the AASHTO Green Book (*3*) presents a discussion of indirect left-turn intersection treatments within Chapter 9 on pages 705 to 709; however, jughandle intersections are only briefly discussed within this section and, with the exception of Green Book Exhibits 9-88 (Figure 62A) and 9-89 (Figure 62B) illustrating near-side and far-side configurations, very little design guidance is offered. In fact, the only information provided on jughandle intersections is that: 1) they should be used at intersections where the median is too narrow to provide a deceleration lane for left-turning vehicles and the traffic volumes, speeds, or both, are relatively high, and 2) the loop ramp configuration should be considered when the right-of-way in the opposite (near-side) quadrants is more expensive or where the far-side quadrants offer improved vertical alignment and similar grading costs.

Furthermore, no guidance is provided in the MUTCD (*31*) regarding recommended signage for a jughandle intersection. Therefore, national design guidance is basically non-existent, and as a result, jughandle intersections do not appear to be used frequently by STAs. A 1996 survey conducted as part of NCHRP Synthesis 225 (*73*) reported that only 16 of the 69 responding state and local agencies had used the jughandle intersection design. A more recent survey of STAs conducted by Maze et al. (*2*) revealed that only 8 of the 28 responding agencies had used the jughandle intersection design at expressway intersections.

TABLE 20. General Advantages/Disadvantages of Jughandle Intersection Design

ADVANTAGES	DISADVANTAGES
1) Meets right-hand exit expectations of expressway drivers. If agencies use multiple jughandle intersections along an arterial corridor, driver confusion would decline, lane changes would decrease, and speeds in the passing lane would increase (*30, 66*). 2) Decreases speed differentials in the expressway passing lane, thereby reducing delay to through expressway traffic (*30, 66, 69, 71*). 3) Reduces total conflict points and spreads them out over a larger area (see Figure 60) (2, *30, 66, 69, 71*). 4) Reduces vehicle paths through the median. 5) Longitudinal left-turn gap selection is replaced by lateral gap selection. 6) Expected to reduce "left-turn leaving" and rear-end crash types (*16, 71*). 7) Eliminates left-turns from the through lanes and provides storage for left-turning traffic at intersections where the median is too narrow to provide left-turn deceleration lanes (*3*). 8) Since direct U-turns and direct left-turns are not allowed from the expressway, the median may be designed narrow (*9, 30, 66*). 9) Because the design permits a narrower median width than is otherwise necessary, less right-of-way is needed along the expressway corridor (*30, 66, 69, 71).* 10) The reduced median width narrows the roadway cross-section, thereby reducing the overall crossing distance for minor road traffic and pedestrians/bicyclists (*71*). 11) Indirect U-turns are easier for large commercial vehicles. 12) At signalized intersections, traffic operations are improved because the jughandle design eliminates the need for a left-turn signal phase on the expressway (*30, 35, 69, 71*). Shorter cycle lengths should be considered to minimize vehicle queues on the minor road (*71*).	1) May increase the frequency of right-angle collisions. 2) More right-of-way is needed in the vicinity of the intersection to accommodate the jughandle design (*2, 30, 64, 66, 69, 71*). Thus, jughandle intersections should be generously spaced so that the extra right-of-way costs do not overwhelm the right-of-way savings along a corridor (*66*); however, intersections should be frequent enough so that minor street crossings are not overloaded with traffic (*30*). 3) Drivers are generally unfamiliar with this design and it may create driver confusion; thus, more signage is necessary to guide drivers through the indirect left-turn (*2, 16, 30, 66, 69, 71*). Typical signs used in New Jersey and Pennsylvania are shown in Figure 61A. Another possible diagrammatic signing option is shown in Figure 61B. 4) Increased travel distance, time, delay, and stops for left turns from the expressway, especially if cross street queues block the ramp terminals (*2, 30, 64, 66, 71*). 5) Because the intersection geometry does not prohibit direct left-turns from the expressway, there is nothing preventing driver disregard of the left-turn prohibitions (*2, 30, 66*). 6) Additional construction, signage, and maintenance costs (*30, 66*). 7) Pedestrian/bicyclist navigation of the intersection becomes more complex as they must cross the ramps and the main intersection. Each additional crossing increases pedestrian/bicyclist exposure to conflicts (*64, 66, 71*).

TABLE 21. Specific Advantages/Disadvantages of Near-Side Jughandle Intersections

ADVANTAGES	DISADVANTAGES
1) Right and left-turning traffic exit the expressway at a common location. 2) Left-turning expressway traffic only passes through the main intersection once. 3) The roadway/ramp for right and left-turning expressway traffic is essentially an offset right-turn lane, thereby improving sight distance at the main intersection for minor road traffic and reducing the potential for "near-side" right-angle collisions. 4) The design reduces the total number of conflict points from 42 to 28 (see Figure 60). 5) Conflicts between vehicles and pedestrians/ bicyclists are reduced at the main intersection because right-turns from the expressway are separated out via the near-side jughandles (*71*).	1) More stops are required of left-turning traffic exiting the expressway as compared with a conventional intersection or a far-side jughandle configuration.

TABLE 22. Specific Advantages/Disadvantages of Far-Side Jughandle Intersections

ADVANTAGES	DISADVANTAGES
1) The design reduces the total number of conflict points from 42 to 26 (see Figure 60). 2) Far-side loop ramps eliminate the need for left-turns onto the cross street from near-side ramps and allow an easier right-turn merge onto the minor road (*30, 66*).	1) Two separate exit points are created for right and left-turning traffic exiting the expressway. 2) Left-turning traffic leaving the expressway must pass through the main intersection twice, increasing the total entering traffic volumes and the opportunity for collisions (*3*). 3) More through capacity is needed on the mainline at the main intersection since left-turning traffic is not filtered out before the intersection (*69*). 4) The travel distances for left-turning traffic exiting the expressway are typically longer with a far-side jughandle configuration as compared with a near-side configuration (*66, 71*). 5) The far-side configuration can cause extra weaving conflicts between loop ramp traffic (entering and exiting the loop) and traffic turning right from the minor road, particularly if there are multiple lanes on the minor road approach and if a separate ramp for right-turning minor road traffic is not provided (*30*). 6) Typically, additional right-of-way is needed to construct a loop ramp as compared with a near-side ramp (*71*).

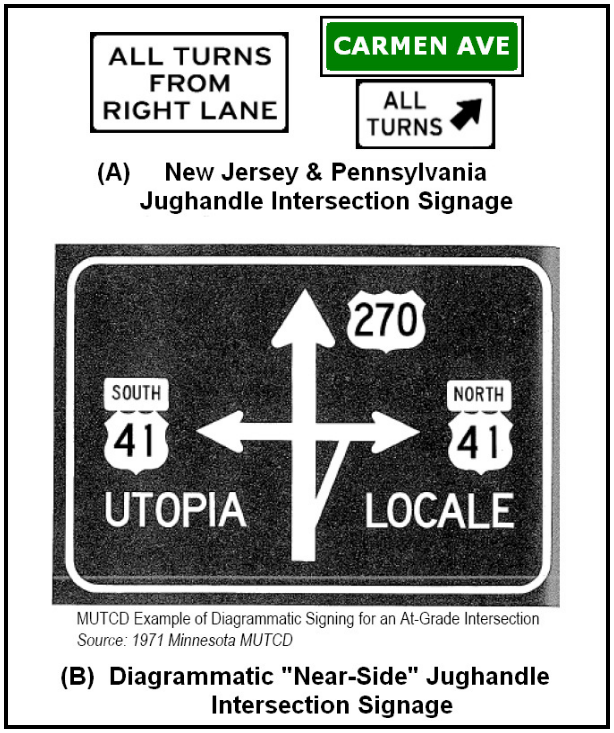


FIGURE 61. Example signage for jughandle intersections

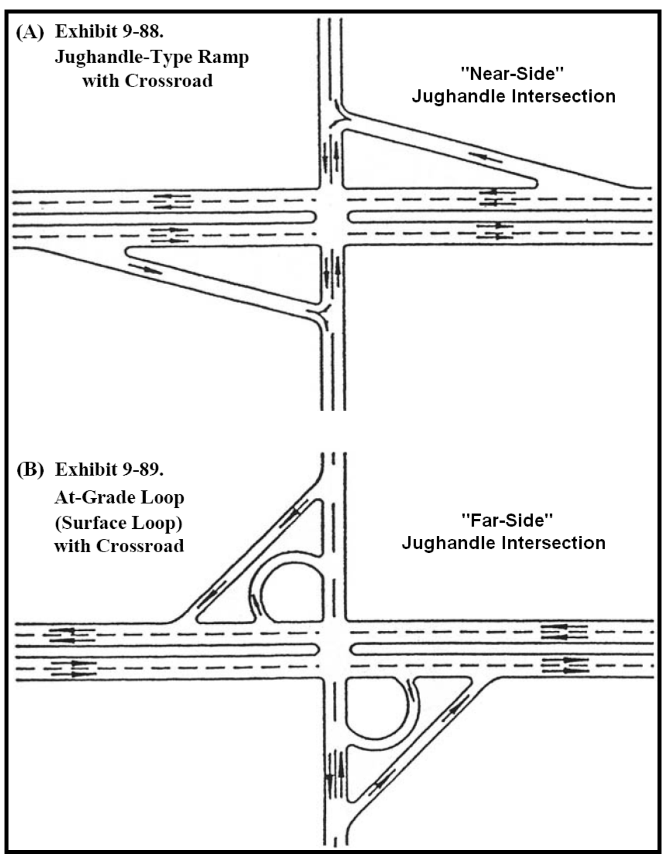


FIGURE 62. Green Book Exhibits 9-88 and 9-89; jughandle intersections (3)

The NJDOT is the most prominent user of the jughandle intersection design, having used it for over 30 years on hundreds of miles of heavy volume arterials, predominantly at signalized intersections (*30, 66*). An aerial photo of one such intersection is shown in Figure 63. Section 6.08 of the NJDOT Roadway Design Manual (*68*) includes jughandle intersection design guidance with respect to access control, design speed, ramp widths, superelevation rates, and cross-slopes. Figures 64 and 65 illustrate NJDOT design standards for near-side and far-side jughandle intersections, respectively. In addition, New Jersey's experience with the jughandle intersection has provided the following basic guidelines for their design as reported by Reid (*30*):

1) There are only a few examples of jughandles located at isolated intersections or along corridors without median barrier; these intersections typically experience higher crash rates,

2) Where jersey barrier controls left-turn access, speed limits that balance facility efficiency and safety are typically 50 or 55 mph,

3) On a four-lane arterial, a minimum two foot offset is desired from the left travel lane to the median barrier; greater separation is desired on a six or eight lane arterial,

4) A minimum distance of 100 feet is required between the ramp terminal and the main intersection (as shown in Figures 64 and 65) to provide at least some room to store vehicles on the minor road without blocking the ramp terminal. In addition, ramps should be long enough to provide adequate storage, thereby preventing queue spillback onto the expressway, and

5) The ability to use shorter signal cycles due to the reduction in signal phases can also reduce the occurrence/frequency of large queues on the minor road.

FIGURE 63. Aerial photo of near-side jughandle intersection (30)

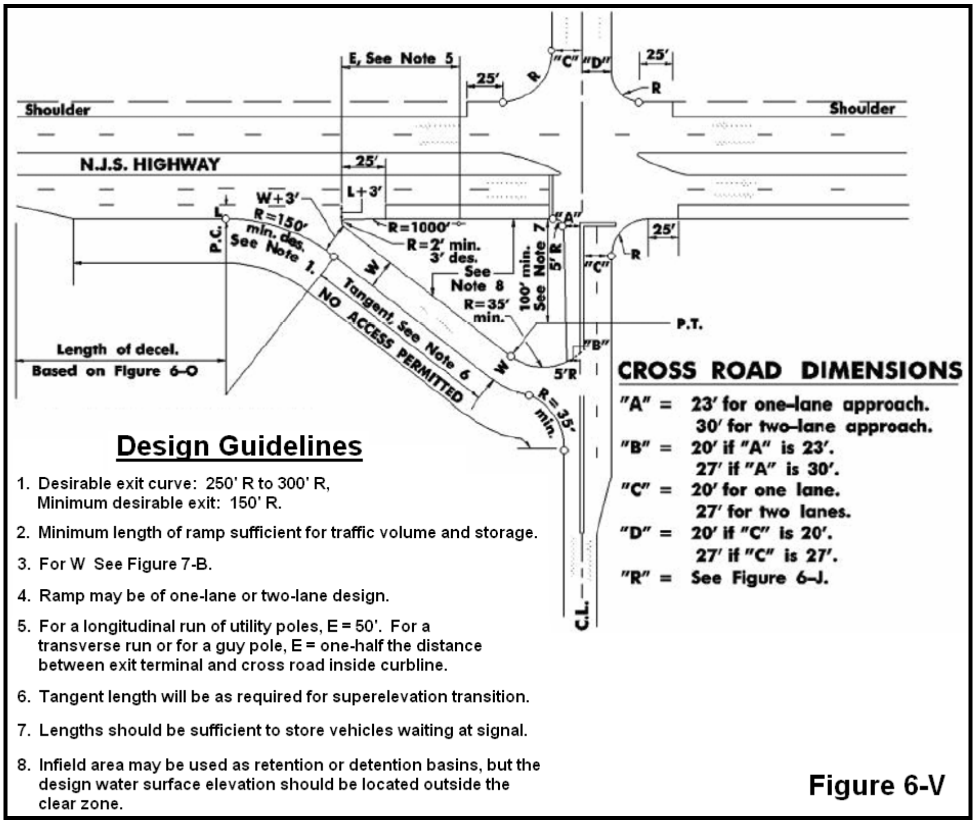


FIGURE 64. NJDOT design standard for near-side jughandle intersection (68)

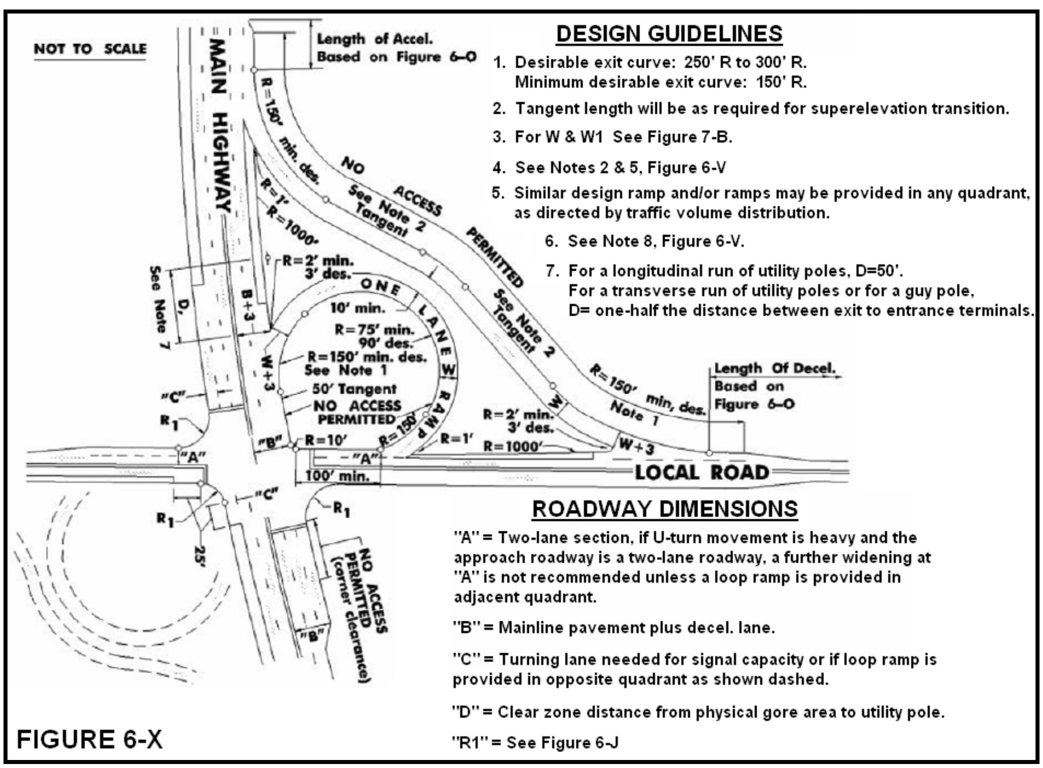


FIGURE 65. NJDOT design standard for far-side jughandle intersection (68)

3.4.3 Case Studies of Implementation & Safety Effectiveness

In the 2004 Maze et al. (*2*) survey, respondents from the following eight STAs indicated that they had used jughandles at expressway intersections: Alabama, California, Missouri, New York, Oregon, Pennsylvania, South Carolina, and Washington. For the purpose of this study, phone interviews were conducted with respondents from each of these STAs as well as with respondents from the 22 STAs that were not included in the Maze et al. (*2*) survey. The purpose of these interviews was to obtain before and after crash data for any conventional TWSC rural expressway intersection which may have been converted into an unsignalized jughandle configuration. In these interviews with the remaining 22 STAs, respondents from Massachusetts, Michigan, and New Jersey indicated that they had used jughandles at expressway intersections. Unfortunately, none of the eleven STAs who indicated that they had used jughandle intersections on expressways were able to provide the requested before and after crash data. The respondents from Alabama and Washington (the same respondents as in the original Maze et al. (*2*) survey) indicated that their responses to the original survey were incorrect and that they do not have any jughandle intersections on their expressway systems. The respondent from Missouri indicated that they have two signalized jughandle intersections near Saint Louis, but that they do not have any unsignalized applications of the design. The respondents from the remaining states (California, Massachusetts, Michigan, New Jersey, New York, Oregon, Pennsylvania, and South Carolina) indicated that they did not have any before-after crash data available for their jughandle intersections. The NJDOT respondent indicated that most of their applications are signalized and that they haven't been constructing many new ones. The ODOT respondent indicated that Oregon has a few unsignalized jughandle intersections on their rural expressway system which were originally constructed on new alignment, thus before crash data does not exist for these intersections. Therefore, due to the lack of data, the safety effectiveness of an unsignalized jughandle intersection was unable to be examined.

3.4.4 Jughandle Intersection Summary

The assumed safety benefit of jughandle intersections is that they reduce rear-end and left-turn related collisions involving vehicles leaving the expressway by eliminating direct left-turns from the mainline and replacing those maneuvers with indirect left-turns via jughandle ramps located on either the near-side or the far-side of the intersection. Although direct left-turns off the expressway are not necessarily restricted via geometry, if traversed correctly, the jughandle intersection designs eliminate roughly 36 percent of the conflict points present at a conventional TWSC expressway intersection. However, because the indirect left-turn movement increases the volume of crossing traffic from the minor road, it is hypothesized that unsignalized jughandle intersections may increase the frequency of right-angle collisions.

Unsignalized jughandle intersections (near-side and far-side) have many advantages and disadvantages in comparison with conventional TWSC rural expressway intersections. However, very few states have used them on rural expressways, national design guidance is lacking, no analytical study of their safety performance was found in the literature review, and a before-after case study was not able to be conducted due to their rarity and the lack of existing crash data. Nonetheless, jughandle intersections seem to have the potential to provide operational and safety benefits when applied in the appropriate situations. For example, on expressways with medians too narrow to provide left-turn lanes, jughandles have clear operational and safety benefits because they are able to keep left-turning traffic from slowing down and/or stopping in the high-speed expressway traffic stream. Therefore, unsignalized jughandle intersections are most appropriate on corridors with narrow right-of-way and at intersections with narrow medians, large volumes of mainline through traffic, and low to moderate left-turn volumes on both the arterial and the intersecting roadway. When used, they should be used throughout an entire corridor to maintain driver expectations.

Clearly, more research is necessary to determine the operational and safety benefits of this intersection design strategy as it applies to unsignalized rural expressway intersections. Until their safety effectiveness is known, jughandle intersections should be used cautiously; however, future implementation is necessary to further evaluate the safety effects and national design guidance is needed. Design details, like those included in the NJDOT Roadway Design Manual (*68*), could be included in the AASHTO Green Book (*3*) along with more detailed guidance indicating when this type of intersection design should be considered and what the tradeoffs are between the near-side and far-side configurations. Furthermore, a typical signing plan for jughandle intersections could be incorporated into the MUTCD (*31*).

3.5 INTERSECTION DECISION SUPPORT (IDS) TECHNOLOGY CASE STUDY

3.5.1 Description

The major theme throughout this entire thesis is that the primary safety issue at TWSC rural expressway intersections is right-angle collisions (far-side right-angle crashes in particular) which are related to the inability of minor road drivers to accurately judge the arrival time of approaching expressway vehicles as they attempt to cross or enter the expressway. NCHRP 500, Volume 5 (*16*) recognizes that providing gap selection assistance to drivers is critical to improving unsignalized intersection safety and using automated real-time information systems to inform drivers when a safe gap exists is one unsignalized intersection safety strategy highlighted in that report. As a result, Intersection Decision Support (IDS) technology is an Intelligent Transportation Systems (ITS) device currently being developed for deployment at TWSC rural intersections to provide gap selection assistance (i.e., aid minor road drivers in judging the adequacy of available gaps in the mainline traffic stream), reduce driver error, and improve intersection safety while avoiding signalization. The IDS technology being developed will utilize radar to track (i.e., detect the presence and speed of) approaching mainline vehicles, computer processors to compute the gap sizes between mainline vehicles, and dynamic message signs which utilize the real-time information to inform minor road drivers when a safe gap exists for crossing or merging with the mainline traffic stream. Such a system should enhance the minor road driver's ability to safely negotiate through TWSC rural expressway intersections.

IDS is a developing technology which began with a research project sponsored by FHWA and a consortium of states (Minnesota, California, and Virginia). The Minnesota team's focus was to develop a better understanding of the causes of crashes at rural unsignalized intersections and then to develop solutions based on those findings. A review of previous research and Minnesota's rural crash records identified poor driver gap selection as a major contributing factor in rural unsignalized intersection crashes (*4*). Historically, the cause of right-angle crashes at TWSC intersections has been classified as either "ran-the-stop" for collisions resulting from a STOP sign violation (i.e., the minor road driver did not stop) or "failure-to-yield right-of-way" for collisions in which the minor road driver stopped, but then collided after proceeding into the intersection (i.e., the selection of an insufficient gap). A study by Najm et al. (*74*) classified approximately 80 percent of TWSC intersection crashes as being related to the selection of insufficient gaps. Other studies have broken down the two crash causation categories further based on driver error. A 1994 study by Chovan et al. (*75*) examined over 100 straight crossing path crashes at TWSC intersections selected from the 1992 Crashworthiness Data System and found that the primary causal factors related to "failure-to-yield right-of-way" collisions were: 1) the driver looked, but did not see the oncoming vehicle (62 percent), 2) the driver misjudged the available gap size or the time-to-arrival of the approaching vehicle (20 percent), 3) the driver had an obstructed view (14 percent), and 4) the roads were ice covered (4 percent). Of these four driver errors, the first three can be described as gap selection issues (either problems involving vehicle detection or judgement of time-to-arrival).

Previous research in Minnesota confirmed these findings. One study identified gap selection as the primary factor contributing to almost 60 percent of the crossing path crashes at rural intersections along two-lane highways in Minnesota (*76*). Additionally, a road safety audit for the US-52 Corridor (a rural expressway between St. Paul and Rochester) identified nine intersections with unusually high crash rates and, at those locations, the fraction of crossing path collisions related to poor gap selection approached 90 percent. Finally, for the initial IDS study, Preston et al. (*4*) performed a detailed review of the crash reports at three expressway intersections over the critical crash rate and found that 87 percent of the right-angle crashes were related to poor gap selection. Moreover, none of the right-angle crashes were a result of minor road drivers running the STOP sign.

3.5.2 Existing Design Guidance

The MUTCD (*31*) has many signs and markings which help a driver recognize that they are approaching a stop-controlled intersection and these devices seem to be effective at TWSC rural expressway intersections given the relatively small proportion of "run-the-stop" crashes. However, based on the over-representation of right-angle "failure-to-yield right-of-way" crashes associated with gap selection at TWSC rural expressway intersections, a primary enhancement to the current MUTCD guidance would be to identify any traffic control devices or markings which would assist minor road drivers in their decision-making process for judging and selecting safe gaps in the expressway traffic stream. Currently, the

MUTCD does not address the need for or the application of such devices and/or markings; therefore, the rural IDS technology is being developed to alert drivers when it is unsafe to enter an intersection.

IDS technology will have three facets: vehicle surveillance instrumentation, a computer processor, and a dynamic message sign. The vehicle surveillance equipment starts with a suite of radar detectors to detect the presence and speeds of vehicles on all approaches. The minor road approaches are also equipped with Light Detection and Ranging (LIDAR) detectors to measure minor road vehicle widths and heights which are later used to determine vehicle types. Knowing the minor road vehicle classification (i.e., passenger cars versus large trucks) is important to understanding each stopped vehicle's acceleration capabilities which affect the size of the minimum gap needed to safely complete a crossing or merging maneuver. Connected to the vehicle surveillance instrumentation is a computer processor which uses the surveillance data to compute the "state" of the intersection. This includes tracking the trajectory (position, speed, and lane of travel) of approaching mainline vehicles, predicting their arrival times at the intersection, computing the size of gaps in mainline traffic, classifying minor road vehicles, computing the minimum required gap, and determining if the actual gap size is larger than the minimum required for safe entry. Finally, a dynamic message sign will be connected to the processor to convey the relevant intersection state data to the minor road drivers, advising them when it is safe to enter the intersection.

Key to the success of the dynamic message sign will be its ability to issue understandable and timely warnings to the minor road driver. Premature warnings will create credibility issues because the system will be viewed as too conservative and the messages will be ignored. Late warnings will do little to reduce crashes as a driver may have already departed the minor road before the warning was activated. Alternate design concepts for the dynamic message sign are currently being evaluated by the Minnesota IDS research team. An initial driver simulator study with younger and older drivers under day and night conditions helped identify general messages that drivers will both understand and comply with (*77*). Some of the initial concepts tested are shown in Figure 66. A second driver simulator study will be completed to help refine the alternatives, finalize the design, and make sure it is compliant with MUTCD (*31*) standards. Following successful driver simulation testing, a field operational test will be conducted with the final approved dynamic message sign.

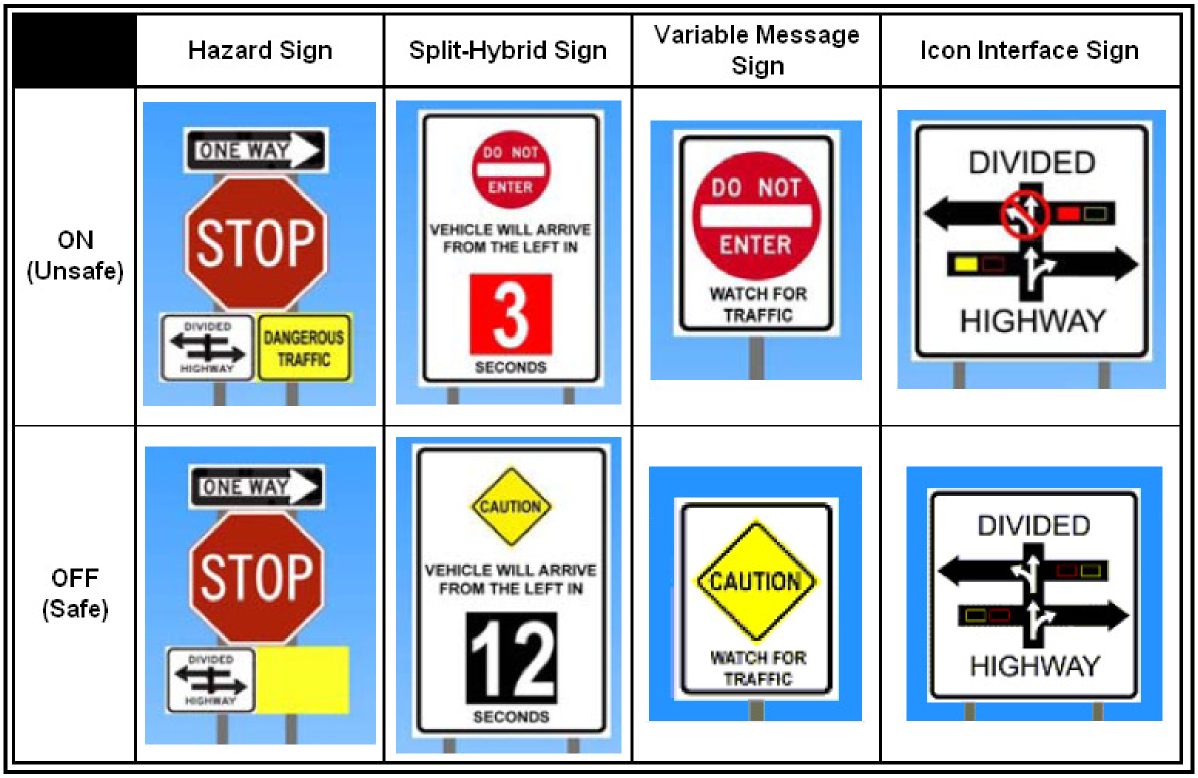


FIGURE 66. Possible IDS changeable message signs (77)

3.5.3 Case Studies of Implementation & Safety Effectiveness

Since the initial study, Mn/DOT and the University of Minnesota initiated a "Rural IDS State Pooled Fund" research project involving eight additional STAs (California, Georgia, Iowa, Michigan, New Hampshire, North Carolina, Nevada, and Wisconsin) to better understand driver gap selection behavior at rural unsignalized intersections across the nation and to develop a nationally deployable IDS system (*78*). The first task of this project involved conducting a comprehensive review of each participating state's crash records to identify candidate locations for future deployment of IDS based on two key crash characteristics: an unusually high crash rate and a high proportion of gap selection related crossing path crashes. A single intersection in each state was then selected for vehicle surveillance instrumentation where gap selection data is currently being collected to analyze: 1) how drivers accept gaps and enter the traffic stream at TWSC rural intersections, 2) whether or not statistically significant regional differences exist in driver gap selection behavior, and 3) how the actual gaps selected compare with the suggested time gaps in the AASHTO Green Book (*3*) which are used for making ISD determinations (*78*). Six of the states (California, Iowa, Minnesota, Nevada, North Carolina, and Wisconsin) selected rural expressway intersections, while the other three states (Georgia, Michigan, and New Hampshire) selected rural undivided highway intersections. Once the IDS technology is ready to be fully deployed, the safety and driver behavior effects of introducing the system at these intersections can be examined.

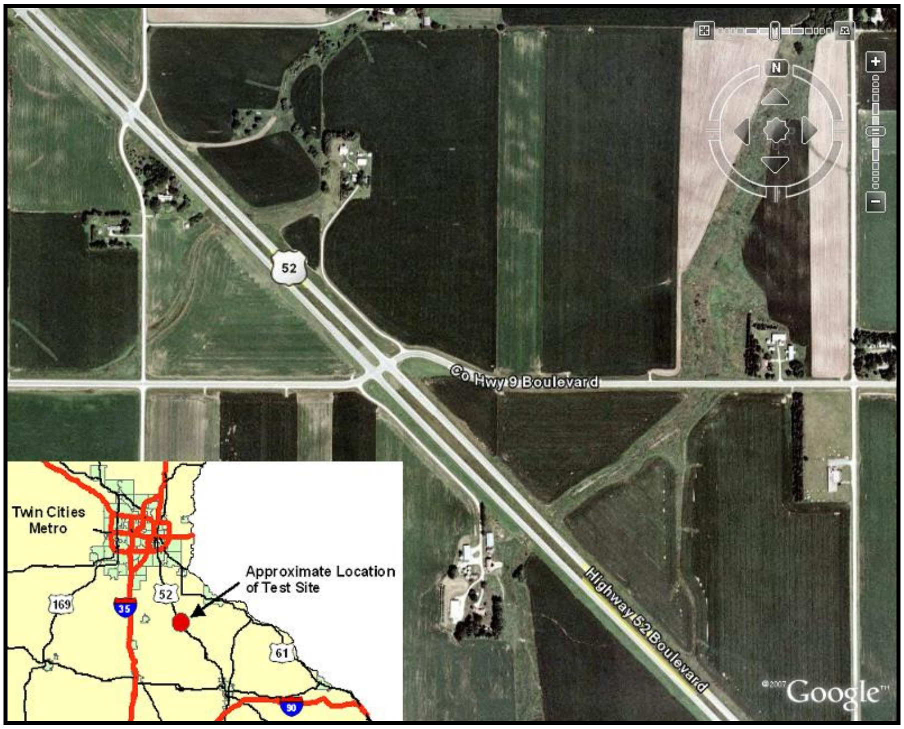


FIGURE 67. Aerial photo of US-52 & CSAH-9

3.5.3.1 Minnesota Experience

In Minnesota, the location selected was the intersection of US-52 and County State Aid Highway 9 (CSAH-9) in Goodhue County. An aerial photo of this intersection is shown in Figure 67. The intersection served as the vehicle surveillance system test bed at which the IDS surveillance instrumentation was first installed. US-52 is a rural expressway with a posted speed limit of 65 mph and CSAH-9 is an undivided paved roadway functionally classified as a rural major collector with a posted speed limit of 55 mph. The intersection is TWSC with guide, warning, and regulatory signage consistent with current MUTCD (*31*) standards. The US-52 approaches have advance junction and guide signs with size, shape, and color consistent with conventional roadways and the beginning of the left and right-turn lanes are identified with LEFT/RIGHT-TURN LANE signs. The CSAH-9 approaches are controlled by STOP signs (ONE WAY and DIVIDED HIGHWAY supplemental signs are mounted with the STOP signs) and include several advance notification signs for the intersection including STOP AHEAD signs and transverse rumble strips. The median at the intersection is controlled by YIELD signs and there is no roadway lighting provided at the intersection.

The 2004 ADT for US-52 in the vicinity of the intersection was approximately 15,500 vpd while the 2003 ADT for CSAH-9 was roughly 925 vpd. Based on these volumes and the Maze et al. (*2*) SPF given in Table 3, this intersection would be expected to experience 3.27 crashes annually. However, between 2000 and 2002, there were a total of 20 collisions at the intersection (0 fatal, 15 injury, and 5 PDO) which equates to 6.67 crashes per year over this time frame. The intersection crash rate was 1.0 crash per mev which is 2.5 times the expected crash rate of 0.40 crashes per mev (*4*). The distribution of crash types at the intersection included 65 percent (13 out of 20) right-angle collisions, which is nearly double the expected 36 percent distribution for TWSC rural expressway intersections in Minnesota (*4*). Review of the investigating officer reports showed that 92 percent (12 out of 13) of the right-angle crashes were far-side collisions related to gap selection, proving that the most hazardous maneuvers at the intersection are indeed the crossing and left-turn movements for drivers on CSAH-9. The Rural IDS Pooled Fund study (*78*) currently underway has found similar crash trends in the participating states where crash analysis has been completed, demonstrating that the problem is not isolated to Minnesota roads.

A field review at the intersection of US-52 and CSAH-9 identified two key geometric issues which may be contributing to the high crash frequency/rate. First, the north and southbound lanes of US-52 have independent vertical alignments (i.e., different elevations as pictured in Figure 68). When US-52 was upgraded from a two-lane undivided highway, design guidelines for driver eye height (when computing sight distance) had changed since the first set of lanes were constructed which is why the independent vertical alignments were originally created. As a result, when a driver is stopped on the eastbound approach of CSAH-9, the elevated southbound lanes of US-52 block their view of northbound traffic (see Figure 68), thereby limiting ISD. However, if the minor road driver uses two-stage gap selection (i.e., stops in the median and looks again), there is sufficient sight distance available to safely complete their crossing or left-turn maneuver.

FIGURE 68. Sight distance issue for eastbound CSAH-9 traffic looking south

A second design issue at the intersection is the horizontal alignment of CSAH-9. The alignments of US-52 and CSAH-9 intersect at a skew angle; thus, reverse horizontal curves were constructed on the CSAH-9 approaches to create a right-angle (90 degree) intersection as shown in Figures 69 and 70.

FIGURE 69. S-curve on CSAH-9 approaches

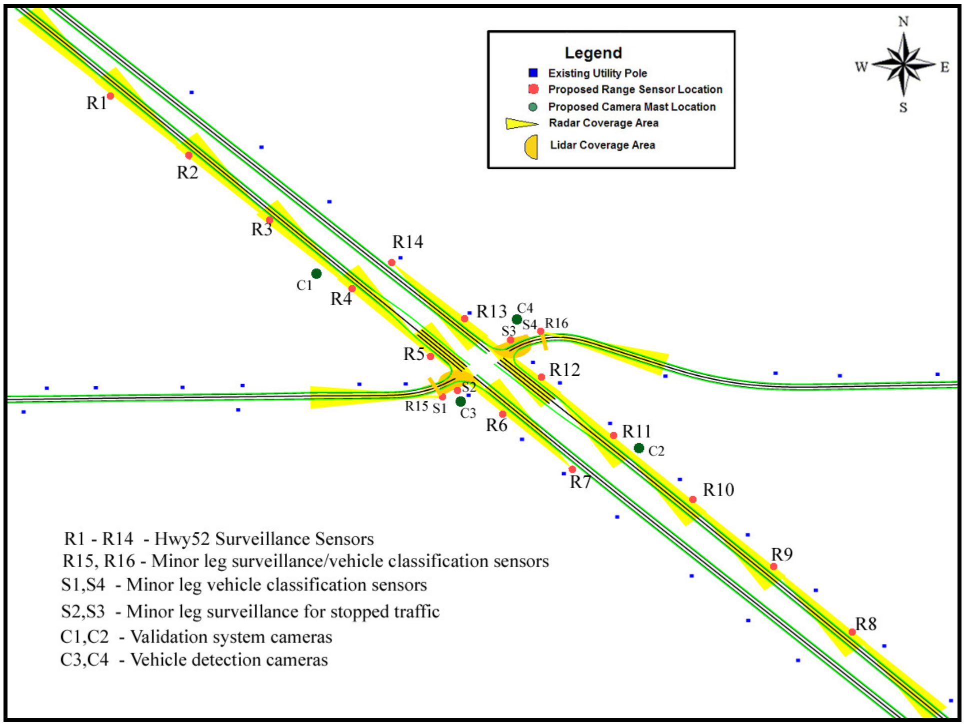


FIGURE 70. Vehicle surveillance system deployed at US-52 & CSAH-9

A corridor study of US-52 recommended upgrading the entire facility to a freeway; however, due to financial constraints, the conversion is expected to take place over a 25 to 30 year period and conversion to an interchange at CSAH-9 is not currently programmed. Consideration was given to installing a traffic signal, but this strategy would adversely affect mobility on US-52; therefore, Mn/DOT was interested in lower cost alternatives and the intersection was selected for future IDS deployment. Vehicle surveillance equipment has been installed at the US-52 and CSAH-9 intersection as shown in Figure 70. A similar, but portable system is currently being moved around the country and deployed at the selected intersections in each of the other eight Pooled Fund States. To date, driver gap selection behavior data has been collected at seven of these intersections. Overall, the vehicle surveillance system has been found to be highly accurate in detecting, tracking, and predicting the arrival times of mainline vehicles. A single radar detector has been found to be over 99.99 percent effective in detecting vehicles (5 misses out of 51,930 radar detections) and the system has also performed exceptionally well in placing vehicles in the correct locations (both laterally and longitudinally within a lane). In addition, the minor road vehicle classification system was found to be over 95 percent accurate.

The test bed deployment in Minnesota cost approximately $250,000 which includes additional infrared and visible light overhead cameras to allow researchers to more closely study collisions and near misses if they happen to occur. Eventually, full deployment of IDS at a rural intersection is hoped to be a cheaper and more effective alternative to signalization.

3.5.4 Intersection Decision Support (IDS) Technology Summary

At TWSC intersections, IDS technology is expected to reduce right-angle collisions by monitoring real-time traffic conditions and informing minor road drivers when a safe gap in mainline traffic exists for making crossing or merging maneuvers. The system is still under development and may not be ready for deployment for a number of years; however, less sophisticated systems used in Virginia, Maine, and Georgia have been deployed at intersections on two-lane roadways and improved safety (*16, 79, 80*). Crash analyses in the participating Rural IDS Pooled Fund States have found candidate locations (6 expressway intersections and 3 undivided highway intersections) for IDS deployment, and when the technology is ready, its safety and driver behavior effects at these sites will be examined. The results of the field testing indicate that the vehicle surveillance instrumentation can accurately detect, classify, and predict the arrival times of vehicles at intersections. As driver gap selection data is collected and analyzed for the Pooled Fund States, a national perspective on driver gap acceptance behavior at rural unsignalized intersections will be obtained. The dynamic message sign for IDS is still being developed and tested in a driver simulator. Initial results have found that the informational content presented on the icon interface and split-hybrid signs in Figure 66 were best understood by drivers and more frequently used to make crossing decisions (*77*). Additional research will be performed to identify the best dynamic sign design, its optimal placement at the intersection, and the sizes of the minimum safe gaps to display before a controlled field experiment is conducted.

The IDS Program has been integrated into the Cooperative Intersection Collision Avoidance Systems Stop Sign Assist (CICAS-SSA) initiative which is pursuing the development of vehicle and infrastructure based cooperative communication systems to warn drivers about likely violations of traffic control devices and gap acceptance crash problems. The CICAS-SSA initiative is a four-year ITS program partnership between the U.S. Department of Transportation (USDOT), STAs, and automobile manufacturers (*81*).

3.6 STATIC ROADSIDE MARKERS CASE STUDY

3.6.1 Description

At rural TWSC expressway intersections, right-angle crashes are a major safety problem (*4*). Common perception is that these crashes occur as a result of minor road drivers not being aware of the intersection and running the STOP sign. Therefore, there are many different techniques used by STAs to help minor road drivers recognize that they are approaching a stop controlled intersection (i.e., STOP AHEAD signs, over-sized/larger STOP signs, multiple STOP signs, transverse rumble strips, intersection lighting, etc.). However, contrary to common perception, right-angle crashes typically occur when the minor road driver first stops at the STOP sign and then drives into the path of a high-speed vehicle on the mainline (*4*). Thus, the primary contributing factor to these crashes seems to be the inability of minor road drivers (either stopped at the stop sign or in the median) to recognize oncoming expressway traffic, to judge their speed and distance (i.e., arrival time), and to select safe gaps in the expressway traffic stream. Consequently, many TWSC rural expressway intersections would benefit from a countermeasure which is able to effectively assist minor road drivers with gap selection; yet, there is no specific treatment found within the MUTCD (*31*) or within the AASHTO Green Book (*3*) designed with this purpose in mind.

The use of static gap selection assistance devices is one experimental technique intended to assist minor road drivers with gap selection at TWSC intersections. The concept involves placing a combination of roadside markers (i.e., delineators, roadway lighting poles, etc.) and/or pavement markings along the mainline to aid minor road drivers waiting at the stop sign in judging whether or not there is adequate clearance to enter the intersection. The markers would delineate a “hazardous” intersection approach zone along the uncontrolled mainline approaches. If an approaching mainline vehicle is within this zone (i.e., closer than the furthest marker), the minor road driver would know that it is not safe (i.e., there is not enough time) to enter the intersection. Thus, the location of the furthest marker identifies the minimum gap necessary for a minor road vehicle to safely enter the intersection. The size, number, and spacing of the delineators and/or pavement markings could be adjusted based on the posted mainline speed limit or based on observed speeds. The devices used to identify the hazardous approach zone (HAZ) could also vary. One proposed strategy is to use pavement markings, such as large "+" symbols, spaced along the mainline approaches (*82*). However, using pavement markings alone is not enough because the markings may not be very visible to the minor road driver, especially during winter months, where the mainline alignment is not straight/flat, or in the far-side expressway lanes. Thus, a second proposal is to use roadside delineators in conjunction with the pavement markings to indicate the location of the HAZ (*82*). An extension of this idea is to use roadway lighting luminaires instead of standard delineators to demarcate the HAZ, with the furthest light pole placed at the gap threshold location. Again, these could be used in combination with pavement markings or other roadside delineators. Using roadway lighting to mark the HAZ at rural intersections with a nighttime crash problem could be especially beneficial since intersection lighting has been found to be effective in reducing nighttime crashes (*46*).

3.6.2 Existing Design Guidance

Currently, the MUTCD (*31*) does not address the need for or the application of such devices and/or markings; however, the Pennsylvania Department of Transportation (PennDOT) has used variations of this signing and marking technique at TWSC intersections on an extremely limited basis. Zwahlen et al. (*82*) presented some design information regarding the PennDOT deployment including lengths of the HAZ based on posted speed limits and recommended pavement marking spacing. A plan view of the design with the mentioned guidance is shown in Figure 71. The HAZ length (or "pattern length" as referred to in Figure 71) was calculated as the distance traveled in five seconds at the posted speed limit plus 80 feet. It is not clear how this calculation was developed, but the values are less than the recommended ISD values given in Green Book Exhibit 9-55 (Figure 72) for passenger cars making a left-turn from stop (ISD Case B1) onto a two-lane highway, which is the most conservative movement to design for.

3.6.3 Case Studies of Implementation & Safety Effectiveness

3.6.3.1 Pennsylvania Experience

As mentioned, PennDOT deployed variations of this static gap assistance treatment at TWSC rural intersections; however, only one location had a specific sign design directing minor road drivers how to use the pavement markings and delineators to improve their gap

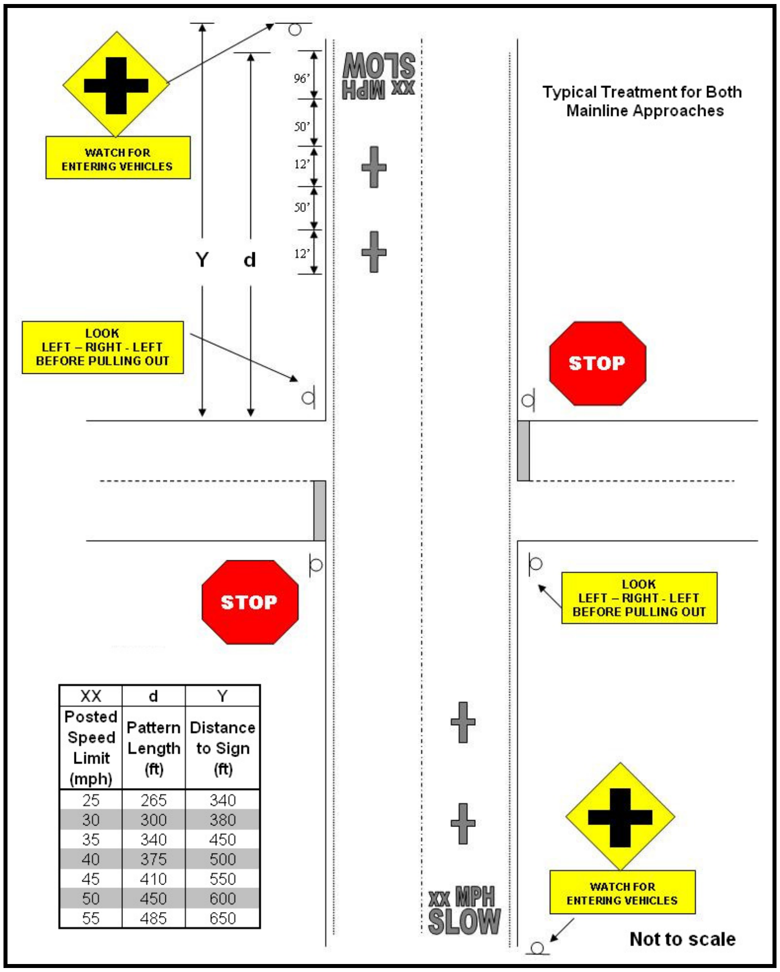


FIGURE 71. Plan view of static gap assistance markings deployed by PennDOT (82)

Metric				US Customary			
		Intersection sight distance for passenger cars				Intersection sight distance for passenger cars	
Design speed (km/h)	Stopping sight distance (m)	Calculated (m)	Design (m)	Design speed (mph)	Stopping sight distance (ft)	Calculated (ft)	Design (ft)
20	20	41.7	45	15	80	165.4	170
30	35	62.6	65	20	115	220.5	225
40	50	83.4	85	25	155	275.6	280
50	65	104.3	105	30	200	330.8	335
60	85	125.1	130	35	250	385.9	390
70	105	146.0	150	40	305	441.0	445
80	130	166.8	170	45	360	496.1	500
90	160	187.7	190	50	425	551.3	555
100	185	208.5	210	55	495	606.4	610
110	220	229.4	230	60	570	661.5	665
120	250	250.2	255	65	645	716.6	720
130	285	271.1	275	70	730	771.8	775
				75	820	826.9	830
				80	910	882.0	885

Note: Intersection sight distance shown is for a stopped passenger car to turn left onto a two-lane highway with no median and grades 3 percent or less. For other conditions, the time gap must be adjusted and required sight distance recalculated.

FIGURE 72. Green Book Exhibit 9-55; design intersection sight distance – case B1 (3)

selection decisions. This application, pictured in Figure 73, was at an unidentified TWSC intersection on a two-lane undivided rural highway. Notice the crosses on the road surface in front of the approaching vehicle, the roadside delineators, and the WAIT IF VEHICLE IN MARKED AREA sign. Other variations of this strategy used similar pavement markings with text indicating the mainline speed limit as shown in Figure 71; however, these designs included a LOOK LEFT-RIGHT-LEFT warning sign on the minor road approaches instead of informing the minor road driver how to use the markers to select a safe gap.

FIGURE 73. PennDOT static gap assistance treatment application (82)

A before-after safety evaluation of the PennDOT pilot program was not conducted. Furthermore, PennDOT stopped using the design illustrated in Figure 73 due to tort liability concerns. One specific situation of concern regarding this application would be if an approaching mainline vehicle were outside the HAZ, but were speeding. Based on the message given on the sign shown in Figure 73, the minor road driver would proceed into the intersection thinking it was safe to do so, but might not actually have enough time to complete their desired maneuver. One possible way to address the tort liability issue in this case would be to revise the message on the instructional sign and place the message on an advisory/warning sign (i.e., black text on a yellow sign) instead of on a regulatory sign (i.e., black text on a white sign) as pictured. Another way to address the issue might be to use roadway lighting to mark the HAZ. Even if the minor road driver doesn't realize the reason for the placement of the furthest light, they would still be more able to see an approaching mainline vehicle at night. The risk of tort liability will also depend on the laws and case histories in individual states. Some states may have laws which are more lenient regarding experimentation with innovative safety strategies as long as the treatment is reasonable and the agency: 1) is responsible with the experimental design, 2) gains the appropriate permissions, 3) continually monitors the treatment, and 4) promptly removes the treatment if there are any unforeseen complications.

Finally, any agency considering the deployment of this strategy should undertake a proactive public education campaign. If the strategy is used in only a few locations, the campaign could be focused locally toward area residents. However, more widespread deployment may require a statewide education campaign (i.e., paid advertisements, updating the state's driver manual, informational brochures given out as drivers renew their license or purchase their vehicle registrations, etc.).

3.6.4 Static Roadside Markers Summary

Static gap selection assistance devices are an experimental approach to aid minor road drivers at TWSC intersections in judging whether or not there is adequate clearance to safely enter an intersection from a stop-controlled approach. The concept involves demarcating a HAZ along the uncontrolled mainline approaches with a combination of roadside markers and/or pavement markings. If a mainline vehicle is in the HAZ, the minor road driver would then know that it is not safe to enter the intersection. The key is communicating this intent to the minor road driver either through a sign placed at the intersection or via a public education campaign. The treatment has only been sparingly deployed and no before-after studies have been conducted to determine its effectiveness in reducing right-angle collisions at TWSC intersections. In order to determine this strategy's actual safety effects, a properly designed before and after crash analysis needs to be conducted. In addition, further research is needed to determine: 1) the length of the HAZ, 2) the optimal type, size, and spacing of roadside markers, and 3) the best way to communicate the intent of the device to the minor road driver. The answers to these research questions may be different under various intersection conditions (i.e., approach speeds, intersection types, etc.).

3.7 LEFT-TURN MEDIAN ACCELERATION LANES CASE STUDY

3.7.1 Description

The greatest crash risk movements (i.e., those accounting for the largest proportion of crashes) at a typical TWSC rural expressway intersection are usually the minor road left-turn and crossing maneuvers (*4*). The underlying problem seems to be that rural expressway intersections present challenges to minor road drivers attempting to select gaps in the expressway traffic stream. One strategy to help left-turning minor road drivers select safe gaps is to provide left-turn median acceleration lanes (MALs). MALs, as illustrated in Figure 74, are auxiliary lanes provided within the median which allow left-turning traffic from the minor road to continue through the median without stopping, accelerate to expressway speed, and gradually merge into the expressway traffic stream.

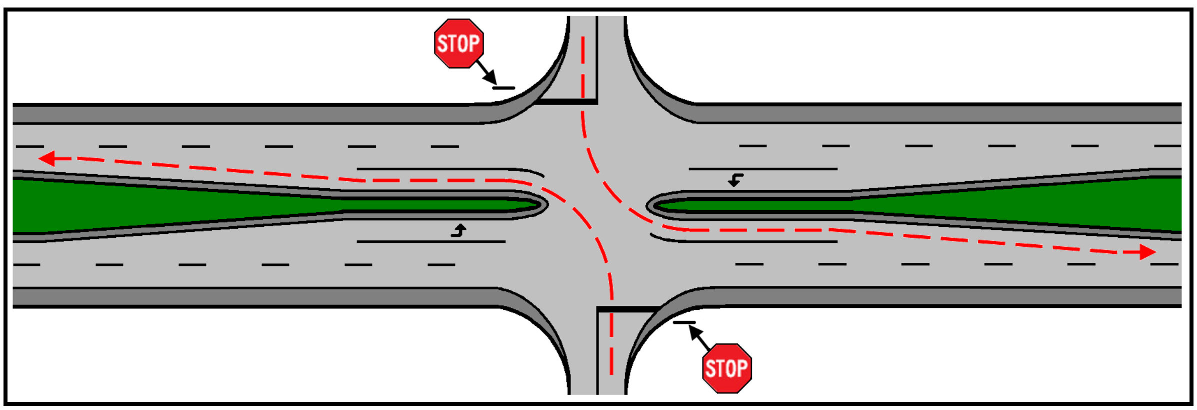


FIGURE 74. Expressway intersection with MALs

MALs are expected to provide several potential safety and operational benefits at rural expressway intersections (*2, 3, 9, 16, 24*). First and foremost, they should make it easier for left-turning minor road drivers to find acceptable gaps in high-speed and/or high volume expressway traffic, thereby increasing safety and reducing delay. MALs do this by allowing left-turning minor road drivers to: 1) cross the near set of expressway lanes without having to simultaneously consider the availability of gaps in the far expressway lanes, 2) use their side/rear-view mirrors to merge, thereby reducing their need to judge gaps at right-angles, and 3) merge with expressway traffic at higher speeds, thereby reducing the size of the critical gap and the required sight distance. In addition, by allowing left-turning vehicles to merge with expressway traffic at high speeds, MALs are expected to reduce speed differentials in the passing lanes of the expressway and allow expressway drivers to better anticipate the presence of entering minor road vehicles. Finally, MALs provide additional median storage for left-turning minor road vehicles, thereby preventing longer left-turn vehicles from encroaching on the near-side through lanes of the expressway while being stored in a narrow median. As a result, MALs are expected to reduce right-angle, rear-end, and sideswipe collisions resulting from conflicts between vehicles turning left onto an expressway and through expressway vehicles. However, MALs will not provide any of these benefits unless they are properly used; therefore, driver education and additional signage/marking may be necessary. A Mn/DOT educational brochure showing how to use MALs is included within the appendix of a study conducted by Hanson (*24*).

The operational and safety effects of MALs have not been widely studied. In 1980, crash rate models developed by Van Maren (*20*) showed that right-angle crash rates at 14 high crash frequency multi-lane divided highway signalized intersections in rural Indiana were reduced with the presence of MALs. In 1985, an ITE Technical Committee (*83*) concluded that MALs appear to promote efficient left-turns onto the major roadway and reduce both crashes and traffic conflicts; however, it was stated that sufficient data were not available for a detailed analysis. In 1995, NCHRP 375 (*9*) conducted field studies at four unsignalized, high-speed (55 mph), divided highway intersections with MALs and concluded that MALs can enhance the operation of intersections on divided highways; however, no quantitative estimates of their safety effectiveness were determined. Finally, in 2002, Hanson (*24*) examined the operational and safety benefits of providing MALs at nine TWSC divided highway intersections in Minnesota. He concluded that MALs substantially reduced median delay and the overall "preventable" crash rate. A closer examination of crash types revealed that MALs reduced rear-end and same-direction sideswipe collisions, but slightly increased right-angle crashes. The increase in right-angle crashes may be due to the fact that the presence of a MAL reduces the amount that opposing mainline left-turn deceleration lanes can be offset. Therefore, providing an operational and safety advantage for left-turns onto a divided highway may create an operational and safety disadvantage for left-turns off of a divided highway (*9*).

3.7.2 Existing Design Guidance

The AASHTO Green Book (*3*) does not provide any design guidance specifically for MALs. However, it does discuss speed-change lanes at intersections within Chapter 9 and on freeways within Chapter 10. When discussing the use of speed-change lanes at intersections, the Green Book states, "Speed-change lanes are warranted on high-speed and on high volume highways where a change in speed is necessary for vehicles entering or leaving the through traffic lanes." However, it goes on to say that warrants for their use cannot be stated definitely and many factors such as speed, volumes, percent trucks, capacity, highway type, desired level-of-service, and the arrangement and frequency of intersections should be considered. On page 689, the Green Book addresses acceleration lanes more specifically stating:

> *Acceleration lanes are not always desirable at stop-controlled intersections where entering drivers can wait for an opportunity to merge without disrupting through traffic. Acceleration lanes are advantageous on roads without stop control and on all high volume roads with stop control where openings between vehicles in the peak-hour traffic streams are infrequent and short.*

In 1995, NCHRP 375 (*9*) recommended that MALs be considered at intersections where adequate median width is available and the following conditions exist:

1) Left-turning minor road traffic merges with high-speed divided highway through traffic,
2) Limited gaps are available in the divided highway traffic stream,
3) There is a significant history of rear-end or sideswipe collisions,
4) ISD is inadequate for left-turning traffic entering the divided highway, and
5) There is a high volume of left-turning trucks (75-100 per day) entering the divided highway.

In 2003, NCHRP 500, Volume 5 (*16*) emphasized that MALs should be considered at unsignalized divided highway intersections where the last three conditions stated above exist. MALs can be used at both three and four-legged intersections; however, their use at four-legged intersections more dramatically alters the conflict patterns within the median (*9*). Further research is necessary to create more specific warrants for MALs. For instance, what mainline volume levels lead to limited gaps or what constitutes a significant history of rear-end/sideswipe collisions?

Once the decision has been made to construct a MAL, the design needs to be determined. The keys in designing a MAL are providing adequate length and creating a median opening area that minimizes conflicts between vehicles entering the MAL and other through/turning vehicles using the median (*16*); however, little design guidance is available in these areas. NCHRP 375 (*9*) stated that MALs are generally constructed as a parallel-type design with an entry taper length of approximately 300 feet at the end of the lane. The AASHTO Green Book (*3*) does not provide specific design guidance for the length of MALs, but does give minimum acceleration lengths for entrance lanes on freeways in Exhibit 10-70 (Figure 18) with adjustments for grade given in Exhibit 10-71 (Figure 19). On page 844, the Green Book states:

> *A speed-change lane should have sufficient length to enable a driver to make the appropriate change in speed in a safe and comfortable manner. Moreover, in the case of an acceleration lane, there should be additional length to permit adjustments in speeds of both through and entering vehicles so that the driver of the entering vehicle can position himself opposite a gap in the through traffic stream and maneuver into it before reaching the end of the acceleration lane.*

The Green Book goes on to say that an acceleration lane length of at least 1,200 feet for a parallel-type entrance is desirable whenever it is anticipated that traffic volumes will reach design capacity in the merging area. However, NCHRP 500, Volume 5 (*16*) cautions that a MAL should not be excessively long because mainline through drivers may mistake it for an additional through lane and be compelled to enter into it.

TABLE 23. Mn/DOT Table 5-4.01A: Desirable Length of Full Width MAL (*84*)

Posted Speed Limit on Divided Highway (mph)	60% of Posted Speed (mph)	Desirable Length of Full Width MAL, Rounded (ft)
45	27	820 *
50	30	990
55	33	1,195
60	36	1,425
65	39	1,670

* Desirable Length = Minimum Length = 820 ft.

Mn/DOT has constructed MALs at rural expressway intersections and Chapter 5 of the Mn/DOT Road Design Manual (*84*) contains a short section on MALs which includes a table (Table 23) for determining the full-width length of a MAL based on the acceleration capabilities of large trucks and the distance required for them to accelerate to 60 percent of the posted speed limit on the divided highway. The desirable lengths of MALs given in Table 23 were calculated using the following equation:

$$S = \frac{V_f^{\,2} - V_i^{\,2}}{2A}$$

where:

S = Minimum calculated length of MAL (ft),

V_f = Speed achieved at the end of distance S (ft/s),

[Mn/DOT calculation, V_f = 60% of posted speed limit on divided highway],

V_i = Initial speed (ft/s), [Mn/DOT calculation, V_i = 0 ft/s],

A = Rate of acceleration (ft/s^2), [Mn/DOT calculation, A = 0.98 ft/s^2],

(Designers may use the acceleration rate of their desired design vehicle).

In 2002, Hanson (*24*) developed more specific design guidelines for minimum lengths of MALs at divided highway intersections where the divided highway speed limit is above 55 mph. These guidelines, shown in Table 24, were developed through observing driver usage of MALs in the field and are ultimately based on the design peak hour volumes in the passing lane of the divided highway.

TABLE 24. Minimum MAL Lengths on High-Speed (> 55 mph) Divided Highways (*24*)

Peak Hour Volume in Passing Lane (vph)	Length of Full Width MAL (ft)
0 – 300	1,000
300 – 450	1,225
> 450	1,395

Besides recommending lengths of MALs, the Mn/DOT Road Design Manual (*84*) also provides a standard plan for MALs, as shown in Figure 75, and provides the following design criteria:

1) The entering throat should be wide enough so that a left-turning truck will not encroach upon through, passing lane traffic (see Note 1 in Figure 75),
2) The lane width should be wide enough to provide an accelerating truck added buffer space in the zone where the speed differential is the greatest (14 feet as shown in Figure 75),
3) When near another crossover, the acceleration lane taper shall end before that crossover's left-turn lane is developed.

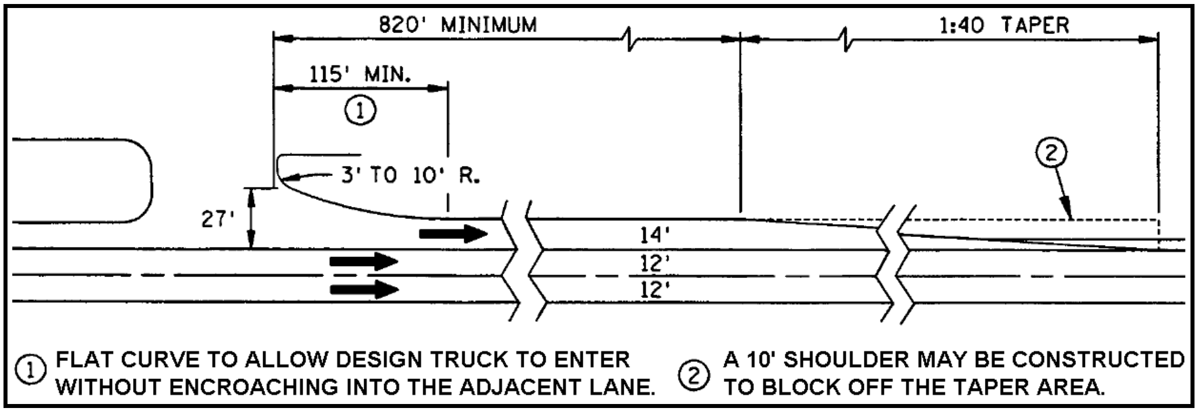


FIGURE 75. Mn/DOT standard plan for MALs (84)

MoDOT has also constructed MALs at rural expressway intersections and Section 233.2.1.19 of their Engineering Policy Guide (*37*) contains a short section on their design. MoDOT's design standards for MALs are similar to Mn/DOT's, with two exceptions. First, in Missouri, the minimum length of a MAL is designed to permit acceleration of trucks to the 85th percentile speed of vehicles operating on the expressway. Second, MoDOT provides a four foot wide shoulder on the median side of the MAL. In addition, although there is no specific policy in place, MoDOT has used the median channelization associated with their Type II median opening (shown in Figure 28) in conjunction with MALs as shown in Figure 76. This channelization may help minimize conflicts between left-turning vehicles entering the MAL and other traffic using the median. It may also help prevent mainline through traffic from entering the MAL by mistake.

FIGURE 76. MoDOT MAL with median channelization

3.7.3 Case Studies of Implementation & Safety Effectiveness

MALs do not appear to be used frequently by STAs. A 1985 survey conducted by ITE (*83*) revealed that only 12 of the 48 responding U.S. transportation agencies (STAs and large municipalities) had constructed MALs. In this survey, the responding agencies stated that they only considered using MALs at high-speed, three-legged divided highway intersections or at intersections with heavy major road volumes where signalization was not warranted, but left-turn entry onto the divided highway was difficult. A more recent survey of 28 STAs administered by Maze et al. (*2*) in 2004 found that only two states, Minnesota and Missouri, have used MALs at TWSC rural expressway intersections.

3.7.3.1 Minnesota Experience

As of 2002, Mn/DOT had constructed MALs at ten expressway intersections. An evaluation conducted by Hanson (*24*) describes the Minnesota experience in great detail. His evaluation examined the effects of MALs in terms of both operations and safety. The operational benefits that MALs provide left-turning vehicles entering a divided highway were examined by conducting a field study at three intersections with MALs and two intersections without MALs. The most evident benefit of providing MALs was reduced median delay, which was measured as the duration of time left-turning vehicles were stopped in the median prior to turning left. At the non-MAL locations, 74 percent of left-turning vehicles experienced median delay and 17 percent waited in the median for more than ten seconds. At MAL intersections, only four percent of left-turning vehicles experienced median delay and only one percent waited in the median longer than ten seconds. If MALs are properly used, no median delay should theoretically occur; therefore, it seems that a small percentage of drivers did not use the MALs correctly.

Hanson (*24*) examined the safety benefits of Minnesota MALs by comparing crash data at nine intersections with MALs versus eight intersections without MALs. The MAL and non-MAL intersections were in close proximity and had similar geometrics and traffic volumes. In comparison, the MAL intersections had a 50 percent lower "preventable" crash rate, a 77 percent lower same-direction sideswipe crash frequency per intersection per year (FIY), a 71 percent lower rear-end crash FIY, and a 15 percent lower right-angle crash FIY. It was also noted that approximately 75 percent of the "preventable" crashes which occurred at the MAL locations were caused by left-turning drivers who did not use the MALs; therefore, the "preventable" crash rate at the MAL locations could have been further reduced if more drivers would have used them properly. Six of the nine MAL intersections had adequate before crash data; therefore, the before period crash data at these six "pre-MAL" sites were compared with the "after" data at the nine MAL intersections. This comparison showed that MALs reduced the "preventable" crash rate by 15 percent. A closer examination by crash type revealed that MALs reduced the rear-end crash FIY by 40 percent, but increased the right-angle crash FIY by 57 percent. The same-direction sideswipe crash FIY was equal at the pre-MAL and MAL sites.

3.7.3.2 Missouri Experience

MoDOT has primarily installed MALs at intersections with large volumes of trucks making left-turns onto divided highways where the median width is not wide enough to accommodate their storage as shown in Figure 77. For this case study, MoDOT provided before and after crash data at two locations where MALs have been installed: 1) US-54 and Business-54/Route W in Miller County, and 2) US-50 and Route MM in Pettis County. Figure 77 shows a picture of the US-50 and Route MM intersection with a large truck utilizing the installed MAL. Aerial photos of both intersections are shown in Figure 78. Both intersections are four-legged intersections located near mainline horizontal curves. At each site, only one MAL was installed. The locations of the MALs are indicated by the red arrows in Figure 78. In addition, the after crash data at each site were limited due to the fact that both intersections were signalized a few years after the MALs were installed.

FIGURE 77. MoDOT MAL at US-50 & Route MM

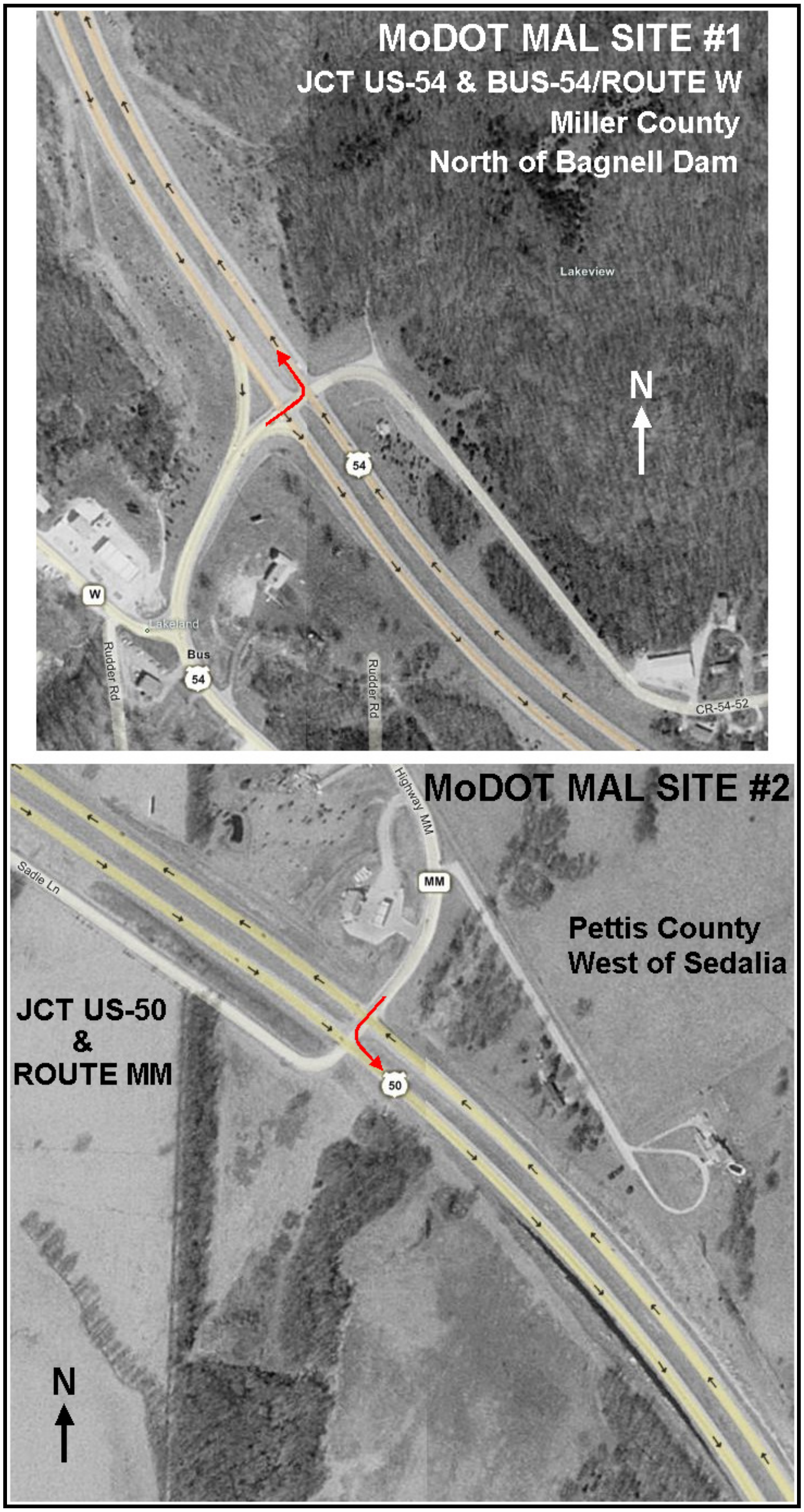


FIGURE 78. Aerial photos of MoDOT MAL study locations

The MAL at US-54 and Business-54/Route W was constructed in the fall of 1998 and the traffic signal was installed in January of 2001; therefore, at this intersection, the before period is four years (1994-1997) and the after period is limited to two years (1999-2000). The before and after data are given and compared in Table 25.

TABLE 25. Before-After Crash Data at MoDOT MAL (US-54 & Bus-54/Route W)

	BEFORE	AFTER	% CHANGE
YEARS	4	2	
TOTAL INTERSECTION-RELATED CRASHES	32	18	
Crash Frequency/Year	8.00	9.00	+ 12.50
FATAL CRASHES	0	1	
Crash Frequency/Year	0	0.50	+ Undefined
INJURY CRASHES	16	7	
Crash Frequency/Year	4.00	3.50	-12.50
PDO CRASHES	16	10	
Crash Frequency/Year	4.00	5.00	+ 25.00
RIGHT-ANGLE/BROADSIDE CRASHES	20	9	
Crash Frequency/Year	5.00	4.50	-10.00
Far-Side Right-Angle	13	4	
Crash Frequency/Year	3.25	2.00	-38.46
Near-Side Right-Angle	7	5	
Crash Frequency/Year	1.75	2.50	+ 42.86
Total Minor Road Left-Turn Related	5	1	
Crash Frequency/Year	1.25	0.50	-60.00
Minor Road Left-Turn Related Where MAL was Installed	4	1	
Crash Frequency/Year	1.00	0.50	-50.00
REAR-END CRASHES	9	2	
Crash Frequency/Year	2.25	1.00	-55.56
SIDESWIPE CRASHES	0	0	
Crash Frequency/Year	0	0	0
OTHER CRASHES	3	7	
Crash Frequency/Year	0.75	3.50	+ 366.67
NOTE: No statistical before-after comparison was performed (< 3 years of after data).			

Since there are only two years of after data, no before-after statistical comparison was conducted for this site. Nevertheless, the raw data show that, although the annual crash frequency increased by 13 percent overall in the after period, the target crash type frequencies were reduced. Right-angle crashes were reduced by 10 percent, far-side right-angle crashes were reduced by 38 percent, minor road left-turn related crashes were reduced by 60 percent, and rear-end crashes were reduced by 56 percent. Since only one MAL was installed at this site, the before and after crash data related to the minor road left-turn movement the MAL was meant to aid (i.e., the eastbound to northbound left-turn) were examined to more precisely determine the safety effects of the MAL. As shown in Table 25, crashes related to the minor road left-turn movement where the MAL was installed were reduced by 50 percent. However, one unexpected result of this study was that near-side right-angle crashes increased in the after period by 43 percent. It was thought that MALs may reduce near-side right-angle collisions due to the fact that they should reduce near-side lane encroachments and allow left-turning minor road drivers to more clearly focus on finding a gap in near-side expressway traffic since they don't have to concern themselves with simultaneously finding a gap in the far-side lanes.

The MAL at US-50 and Route MM was constructed in June of 2000 for a cost of approximately $218,000; however, this price-tag also included the construction of a right-turn lane for westbound traffic on US-50 turning northward onto Route MM which was completed at the same time. The traffic signal at this location was installed in August of 2003; therefore, at this intersection, the before period is seven years (1993-1999) and the after period is limited to three years (July 1, 2000 through June 30, 2003). At this site, there were 264 left-turns from southbound Route MM toward the east during the PM peak with 20 percent trucks making this movement which is why the MAL was constructed. The before and after crash data are given and compared in Table 26.

Overall, total crashes were reduced by 23 percent in the after period and most of the target crash types were reduced as well. Right-angle crashes were reduced by 25 percent, far-side right-angle crashes were reduced by 48 percent, near-side right-angle crashes were reduced by 20 percent, and rear-end crashes were reduced by 79 percent. However, minor road left-turn related crashes where the MAL was installed (i.e., collisions involving left-turns from southbound Route MM) increased by 17 percent which was unexpected since this is the main crash type being targeted by the MAL treatment. With at least three years of before and after data available at this site, statistical comparison of the mean annual crash frequencies was performed. Using a one-tailed t-test for detecting differences in sample means assuming unequal variances and a 90 percent level of confidence ($\alpha = 0.10$), the mean annual crash frequency was significantly reduced in the after period for injury and rear-end crashes. However, the reduction in rear-end crashes may also be due to the installation of the right-turn lane at this site. In addition, the frequency of sideswipe crashes significantly increased in the after period.

TABLE 26. Before-After Crash Data at MoDOT MAL (US-50 & Route MM)

	BEFORE	AFTER	% CHANGE	SIGNIFICANT DIFFERENCE AT
YEARS	7	3		
TOTAL CRASHES	64	21		
Crash Frequency/Year	9.14	7.00	-23.44	α = 0.1541
FATAL CRASHES	3	2		
Crash Frequency/Year	0.43	0.67	+ 55.56	α = 0.3084
INJURY CRASHES	34	8		
Crash Frequency/Year	4.86	2.67	-45.10	α = 0.0682 *
PDO CRASHES	27	11		
Crash Frequency/Year	3.86	3.67	-4.94	α = 0.4563
RIGHT-ANGLE/BROADSIDE CRASHES	50	16		
Crash Frequency/Year	7.14	5.33	-25.33	α = 0.1665
Far-Side Right-Angle	9	2		
Crash Frequency/Year	1.29	0.67	-48.15	α = 0.2571
Near-Side Right-Angle	41	14		
Crash Frequency/Year	5.86	4.67	-20.33	α = 0.2184
Total Minor Road Left-Turn Related	6	3		
Crash Frequency/Year	0.86	1.00	+ 16.67	α = 0.4270
Minor Road Left-Turn Related Where MAL was Installed	6	3		
Crash Frequency/Year	0.86	1.00	+ 16.67	α = 0.4270
REAR-END CRASHES	11	1		
Crash Frequency/Year	1.57	0.33	-78.79	α = 0.0336 *
SIDESWIPE CRASHES	1	3		
Crash Frequency/Year	0.14	1.00	+ 600.00	α = 0.0005 *
OTHER CRASHES	2	1		
Crash Frequency/Year	0.29	0.33	+ 16.67	α = 0.4542

* Significant difference in sample means assuming unequal variances at a 90% level of confidence using a one-tailed t-test.

In studying the crash data at this site, the intersection had a clear over-representation of near-side right-angle crashes, which is not necessarily the target crash type MALs are meant to address. In the before period, 82 percent of the right-angle crashes (41 out of 50) were near-side collisions and, while the near-side right-angle crash frequency decreased by 20 percent in the after period, near-side collisions still constituted 87.5 percent of all right-angle collisions (14 out of 16) after the MAL installation. A majority of the near-side right-angle collisions occurring at this site involved vehicles traveling southbound on Route MM (it is not clear whether they were turning left or crossing) colliding with westbound traffic on US-50. The reasons for this trend are unclear; however, it may be linked to a combination of the horizontal curve located on the westbound US-50 approach and the number of large trucks turning right from that same approach. This is where the right-turn lane was installed; however it was not an offset right-turn lane. Figure 79 clearly shows how the presence of a large right-turning truck and the horizontal curve on westbound US-50 combine to obstruct a southbound driver's view of approaching expressway traffic in the near-side lanes. Therefore, installing an offset right-turn lane could have been another option to improve safety at this intersection (see the "Offset Right-Turn Lane Case Study" presented next).

FIGURE 79. Obstructed view of southbound driver on Route MM looking east at US-50

3.7.4 Left-Turn Median Acceleration Lanes Summary

The assumed safety benefit of MALs at TWSC rural expressway intersections is that they reduce the potential for right-angle collisions (particularly far-side right-angle collisions), as well as rear-end and sideswipe collisions in the far-side expressway lanes related to minor road traffic turning left onto a divided highway. MALs accomplish this by allowing left-turning minor road drivers to continue through the median without stopping, accelerate to expressway speed, and then gradually merge into the expressway traffic stream using their rear-view mirrors, consequently making it easier to select gaps in high-speed and/or high-volume expressway traffic. MALs may also help reduce near-side right-angle collisions by reducing the opportunity for near-side through lane encroachments and allowing left-turning minor road drivers to focus their attention towards oncoming traffic in the near set of expressway lanes. TWSC rural expressway intersections expected to benefit from MALs include those with: 1) a history of crashes involving left-turning minor road vehicles, 2) limited gaps available in the far-side expressway lanes, 3) large volumes of trucks entering the expressway via left-turns, 4) narrow medians unable to store large trucks, and 5) inadequate intersection sight lines to the far-side expressway lanes for left-turning traffic entering the expressway (see Figure 68). Limited experience with MALs in Minnesota and Missouri has shown that the concept can offer improved safety performance for left-turning traffic entering a divided highway. Table 27 summarizes these results for the target crash types. Right-angle, far-side right-angle, and rear-end collisions were all reduced in the presence of MALs at both Missouri sites examined in this case study; however, no definitive conclusions regarding the safety effects of MALs can be drawn from this limited study.

TABLE 27. MAL Safety Effectiveness Summary

	LOCATION		
ANNUAL CRASH FREQUENCY	**MINNESOTA STUDY (*24*) (% CHANGE)**	**MoDOT US-54 & BUS-54/ ROUTE W (% CHANGE)**	**MoDOT US-50 & ROUTE MM (% CHANGE)**
Total Crashes	No Data	+ 13	-23
Right-Angle Crashes	+57 and -15	- 10	-25
Far-Side Right-Angle Crashes	No Data	-38	-48
Near-Side Right-Angle Crashes	No Data	+ 43	-20
Minor Road Left-Turn at MAL	-15 and -50	-50	+17
Rear-End	-40 and -71	-56	-79 *
Sideswipe	0 and -77	0	+600 *
* Statistically significant change at 90% confidence level using one-tailed t-test.			

The keys to successfully designing a MAL are providing adequate length and creating a median opening area that minimizes conflicts between vehicles entering the MAL and other vehicles using the median. Mn/DOT and MoDOT provide some design guidance in this regard; however, national design guidance is needed and could be incorporated into the AASHTO Green Book (*3*). Furthermore, a typical signing and marking plan for MALs could be included in the MUTCD (*31*).

3.8 OFFSET RIGHT-TURN LANES CASE STUDY

3.8.1 Description

The purpose of providing exclusive right-turn lanes on expressway intersection mainline approaches is to remove the deceleration and storage of right-turning vehicles from the high-speed through traffic lanes, thereby enabling through traffic to pass by with little conflict or delay and improving the overall safety and capacity of the intersection (*3*). It is generally thought that the presence of exclusive right-turn lanes on the divided highway contributes to intersection safety by reducing speed differentials in the through lanes, consequently diminishing the potential for rear-end collisions, particularly on high-speed, high-volume approaches where right-turn volumes are substantial. However, the limited research assessing the safety effects of providing exclusive right-turn lanes at rural expressway intersections reveals that conventional right-turn lanes may actually increase crashes (*2, 20*).

A crash model developed by Van Maren (*20*) in 1980 for 39 randomly selected, multi-lane divided highway intersections in rural Indiana showed that intersection crash rates increased with the presence of a right-turn deceleration lane on the divided highway. In a more recent study, a rural expressway intersection SPF developed by Maze et al. (*2*) using 644 TWSC expressway intersections in rural Iowa revealed a similar trend (this particular SPF is not the same one given in Table 3). Although this result was statistically significant at a 90 percent level of confidence, the authors speculated that the higher crash rates at locations with right-turn lanes may not have been directly linked to their presence, but were instead a result of the fact that right-turn lanes had been installed at high crash locations. However, another explanation of these findings might be the fact that vehicles using a conventional right-turn lane to exit the expressway obstruct the adjacent minor road driver's view of oncoming expressway traffic, thus leading to an increase in near-side right-angle collisions. This condition is illustrated in Figure 79, with a plan view shown in Figure 80A. The substantial increase in trucks and sport utility vehicles (which are more difficult to "see around") in today's vehicle mix, combined with an increase in elderly drivers (who tend to have more difficulty with gap selection) makes this condition more of a concern today than it has ever been in the past.

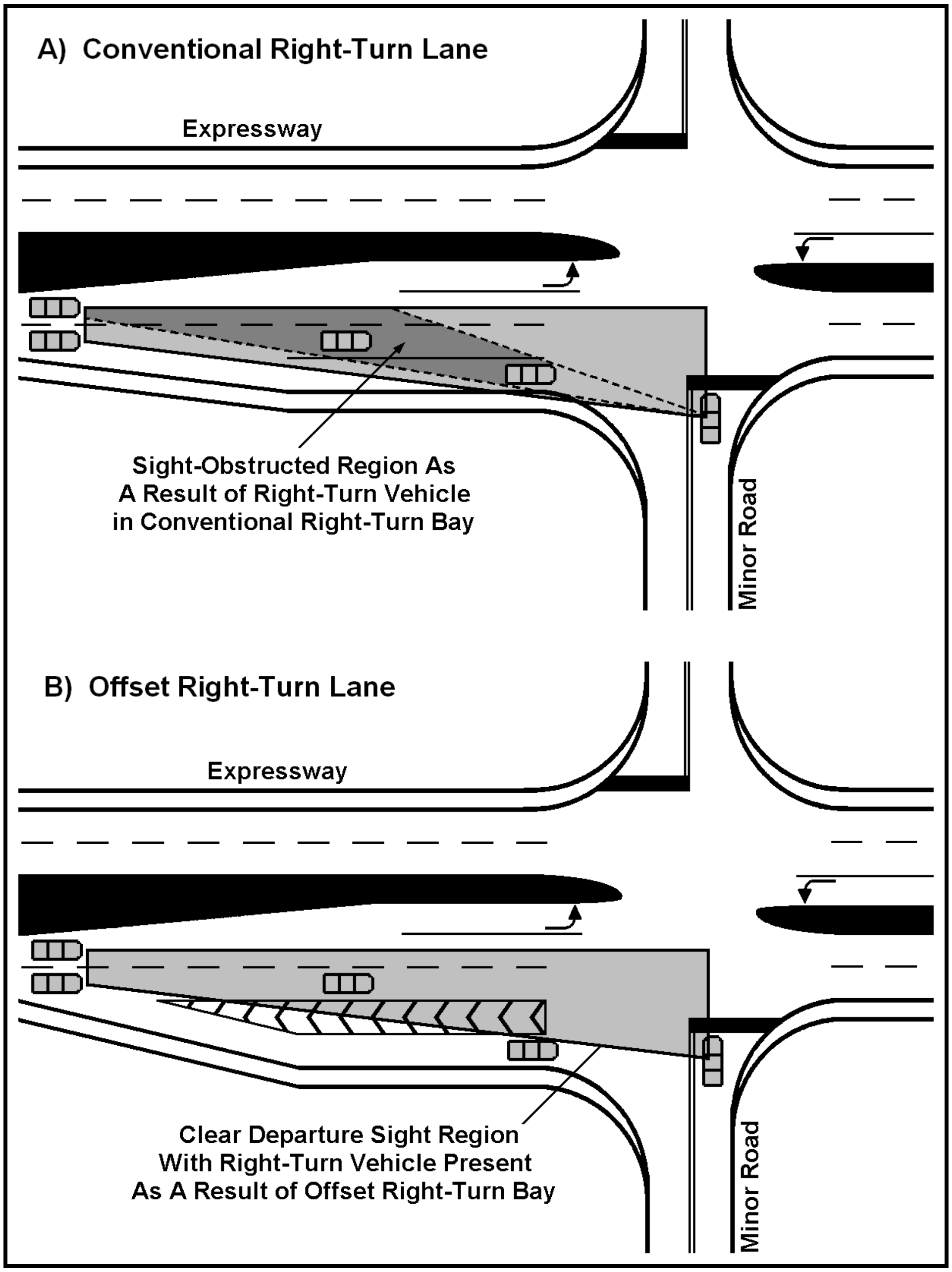


FIGURE 80. Offset right-turn lane design concept illustration

The offset right-turn lane design alternative illustrated in Figure 80B helps to alleviate the sight distance obstruction created by the presence of right-turning vehicles in a conventional right-turn lane. In this design, the mainline right-turn lane is moved laterally to the right as far as necessary so that right-turning mainline vehicles no longer obstruct the view of minor road drivers positioned at the adjacent stop bar. Offset right-turn lanes should improve rural expressway intersection safety by enhancing ISD and making it easier for minor road drivers to select safe gaps in the near-side expressway traffic stream when right-turning expressway vehicles are present. As such, offset right-turn lanes are expected to reduce near-side right-angle collisions between vehicles turning or crossing from the minor road and through vehicles on the divided highway; however, no research has been conducted to determine the safety benefits of applying this strategy at rural expressway intersections (*16*).

3.8.2 Existing Design Guidance

No guidance on the use or design of offset right-turn lanes is currently available in the AASHTO Green Book (*3*). As such, offset right-turn lanes do not appear to be used frequently by STAs. A recent survey of STAs conducted by Maze et al. (*2*) revealed that only 5 of the 28 responding agencies (California, Colorado, Iowa, Oregon, and Washington) had used offset right-turn lanes as a corrective measure at rural expressway intersections. However, design guidance for offset right-turn lanes does appear in Chapter 6C-5 of Iowa's Highway Design Manual (*85*) which states, "When right-turn lane warrants are met, offset (tapered) lanes may be considered in areas where sightline difficulties may occur, such as at the base of a long or steep decline (grade = 5 percent or larger) or at the crest of a hill with a minimum K-value." The design specifications for offset right-turn lanes presented in the Iowa Design Manual are shown in Figure 81. The figure depicts a 30:1 taper and a 20 foot offset with the lane length based on the posted speed limit. However, according to the IA-DOT Design Methods Division, the critical dimension is the 20 foot offset and the taper ratio is modified accordingly based on the selected lane length. More recently however, the IA-DOT has been building offset right-turn lanes by paving a 16 foot wide parallel right-turn area and striping off a 6 foot offset, thus creating a 10 foot wide, parallel offset right-turn lane.

The most important design aspect of an offset right-turn lane is that it should provide the minor road driver with a clear departure sight triangle to the left (i.e., sufficient sight distance along the near-side expressway lanes) when right-turning vehicles are present on the mainline. Meeting this design criterion should aid minor road drivers in judging the suitability of available gaps in the near-side expressway traffic stream when making turning or crossing maneuvers. The recommended dimensions for the legs of a clear departure sight

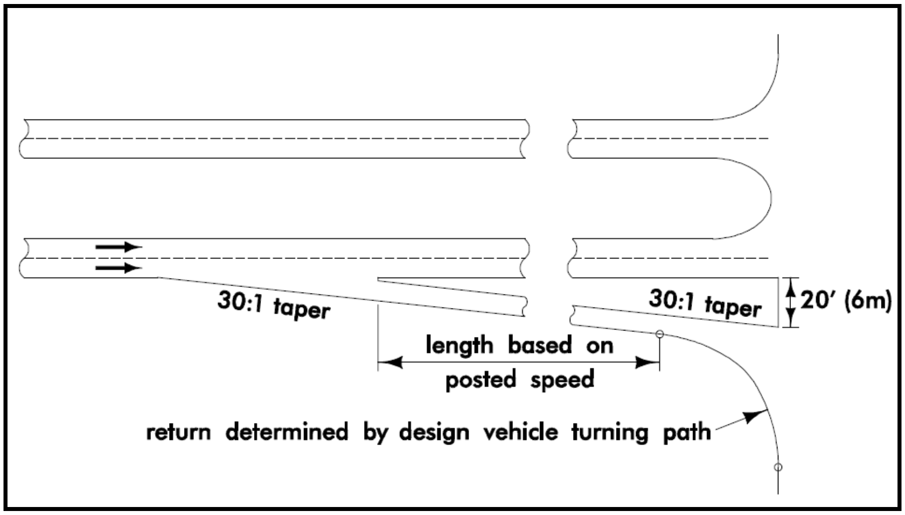


FIGURE 81. IA-DOT standard plan for offset right-turn lane (85)

triangle are described in Chapter 9 of the Green Book (*3*). The required offset distance may vary from intersection to intersection based on each intersection's unique geometry (skew, horizontal curvature, approach grades, design speeds, stop bar placement, etc.); therefore, intersection design plans should be checked to ensure that adequate ISD is provided. Zeidan and McCoy (*86*) provide an example of how this should be done. In 2000, they determined the available sight distance (ASD) to the left for a minor road driver waiting to enter an arterial from a driveway when a right-turning vehicle is present in a right-turn lane as a function of vehicle positions and intersection geometrics. Figure 82 depicts the trigonometry and defines the variables used in their calculations. Tables 28 and 29 present the minimum right-turn lane offsets determined by Zeidan and McCoy (*86*) required to provide adequate ISD based on the design speed of the arterial, the stopping location of the driveway vehicle, and the overall throat width of the driveway. Table 28 assumes a passenger car is the obstructing right-turn vehicle while Table 29 assumes a single unit truck is the obstructing vehicle. Both tables assume right-angle driveway intersections on tangent sections of the arterial; however, the values given in these tables were calculated using ISD standards from the 1994 edition of the AASHTO Green Book and need to be reexamined using the updated ISD criteria given in the current edition (*3*). A sensitivity analysis conducted by Zeidan and McCoy (*86*) showed that adjusting the stopping location of the minor road vehicle and/or the right-turn lane offset distance are the most practical means of providing adequate ISD in this situation.

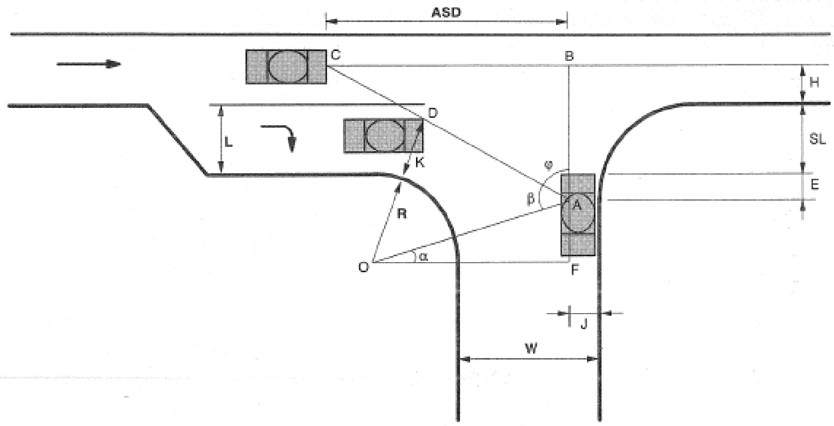


FIGURE 1 ASD. A = location of driver's eye; AB = perpendicular offset from center of near lane to driver's eye; AC = line of sight; BC = centerline of near lane; D = point of tangency between line of sight and outside of right-turn lane vehicle's turning path; E = distance between driver's eye and front of driveway vehicle; H = distance between edge of near lane and center of near lane; J = distance between driver's eye and right side of driveway; K = radial distance between outside edge of turning path and curb; L = width of right-turn lane; O = radius point of right-turn lane curb return radius and turning path; R = curb return radius of right-turn lane; SL = distance between front of driveway vehicle and edge of near lane; W = driveway throat width.

FIGURE 82. Minimum right-turn offset calculation trigonometry diagram (86)

TABLE 28. Minimum Right-Turn Lane Offset; Obstructing Veh. = Passenger Car (*86*)

Design Speed (km/hr)	Minimum Right-Turn Lane Offset (m) assuming 3.6 m right-turn lane					
	Undivided Driveway [a]			Divided Driveway [b]		
	[c] SL = 0	SL = 1 m	SL = 3 m	SL = 0	SL = 1 m	SL = 3 m
60	2.2	3.1	5.0	1.8	2.6	4.3
70	2.3	3.2	5.1	2.0	2.8	4.6
80	2.4	3.3	5.2	2.1	3.0	4.8
90	2.4	3.4	5.3	2.2	3.1	4.9
100	2.5	3.4	5.4	2.3	3.2	5.0
UASD [d]	2.6	3.6	5.6	2.6	3.6	5.6

[a] Undivided driveway with 7.6 m throat (W = 7.6 m).
[b] Divided driveway with 22 m throat (W = 22 m).
[c] SL = Distance driveway vehicle stops from the edge of the roadway.
[d] Available sight distance is unrestricted.

TABLE 29. Minimum Right-Turn Lane Offset; Obstr. Veh. = Single Unit Truck (*86*)

Design Speed (km/hr)	Minimum Right-Turn Lane Offset (m) assuming 3.6 m right-turn lane					
	Undivided Driveway [a]			Divided Driveway [b]		
	[c] SL = 0	SL = 1 m	SL = 3 m	SL = 0	SL = 1 m	SL = 3 m
60	3.8	4.7	6.6	3.4	4.2	5.9
70	3.9	4.8	6.7	3.6	4.4	6.2
80	4.0	4.9	6.8	3.7	4.6	6.4
90	4.1	5.0	6.9	3.8	4.7	6.6
100	4.1	5.0	7.0	3.9	4.8	6.7
UASD [d]	4.2	5.2	7.2	4.2	5.2	7.2

[a] Undivided driveway with 7.6 m throat (W = 7.6 m).
[b] Divided driveway with 22 m throat (W = 22 m).
[c] SL = Distance driveway vehicle stops from the edge of the roadway.
[d] Available sight distance is unrestricted.

3.8.3 Case Studies of Implementation & Safety Effectiveness

Examples of offset right-turn lane implementation at TWSC rural expressway intersections were found in Iowa and Nebraska. These case studies are presented herein.

3.8.3.1 Iowa Experience

Two examples of offset right-turn lane installations on rural expressways were found in Iowa. The first example is located at the intersection of US-61 and Hershey Road near the western edge of Muscatine, IA. An aerial photo of this intersection is shown within Figure 83. US-61 through this area was originally built to expressway standards in 1984. The construction of the US-61/Hershey Road intersection that resulted from this project did not provide any right-turn lanes for vehicles exiting US-61. The intersection remained this way until July of 2003, when offset right-turn lanes were installed on both the northbound and southbound US-61 approaches. Photographs of each offset right-turn lane are shown in Figure 83. During both the before and after periods, the TWSC at the intersection was reinforced with an ICB. Subsequently, the intersection was signalized in November of 2005 and remains that way today; however, this intersection has been a consistent safety problem and is likely to be converted to an interchange sometime in the near future.

FIGURE 83. Offset right-turn lanes at US-61 & Hershey Road – Muscatine, IA

Before and after crash data for this offset right-turn lane installation were obtained from ITSDS and are shown in Table 30. In the 3.5 year before period (1/1/2000 through 6/30/2003) the intersection experienced a total of 15 intersection-related crashes (1 fatal, 10 injury, and 4 PDO), giving an average crash frequency of 4.3 crashes per year, which is about what would be expected based on the entering traffic volumes (*2*). The before period crash rate was 0.95 crashes per mev. In the 2.25 year after period (8/1/2003 through 10/31/2005), there were a total of 11 intersection-related crashes (2 fatal, 5 injury, and 4 PDO), resulting in an average of 4.9 crashes per year (a 14 percent increase which is what was expected based on the change in entering volumes). The after period crash rate was 0.97 crashes per mev, giving an overall crash rate increase of approximately 2 percent. However, in order to gauge the true effectiveness of the offset right-turn lane installation, a closer examination of the crash types targeted by the improvement, namely near-side right-angle collisions, is necessary (*16*).

Because offset right-turn lanes are meant to reduce near-side right-angle collisions, and because offset right-turn lanes were installed on both mainline approaches at this location, a before-after comparison of total near-side right-angle collisions was conducted. As shown in Table 30, the intersection averaged 1.71 near-side right-angle collisions annually in the before period with a near-side right-angle crash rate of 0.38 per mev. In the after period, the site averaged 2.67 near-side right-angle collisions per year with a near-side right-angle crash rate of 0.53 per mev; increases of approximately 56 and 39 percent, respectively. In addition, it appears that after the installation of the offset right-turn lanes, the distribution of far-side to near-side right-angle collisions switched in favor of near-side crashes, which is an unexpected outcome. Overall, the site averaged approximately 4 right-angle crashes annually in both the before and after periods with a right-angle crash rate of approximately 0.80 per mev; however, after installation of the offset right-turn lanes, the distribution of near-side right-angle crashes increased from 46 to 67 percent.

Because two offset right-turn lanes were installed at this location, one on northbound US-61 and one on southbound US-61, a separate before-after comparison of near-side right-angle collisions was conducted for each offset right-turn lane. The results of this analysis are shown in Table 30. Table 30 shows that neither offset right-turn lane was effective in reducing the frequency or rate of near-side right-angle collisions. In both the before and after

TABLE 30. Offset Right-Turn Lane Before-After Comparison (US-61 & Hershey Rd)

	BEFORE	AFTER	% CHANGE ***
ESTIMATED ENTERING AADT (US-61) *	10,000	11,300	+ 13.00
ESTIMATED ENTERING AADT (Hershey Road) *	2,305	2,450	+ 6.29
ESTIMATED TOTAL ENTERING AADT (vpd)	12,305	13,750	+ 11.74
ESTIMATED ENTERING AADT (SB & EB Traffic) *	6,550	7,225	+ 10.31
ESTIMATED ENTERING AADT (NB & WB Traffic) *	5,755	6,525	+ 13.38
EXPECTED CRASH FREQUENCY/YEAR **	4.44	5.04	+ 13.42
YEARS	3.50	2.25	
TOTAL INTERSECTION-RELATED CRASHES	15	11	
Crash Frequency/Year	4.29	4.89	+ 14.07
Crash Rate/mev	0.95	0.97	+ 2.09
FATAL CRASHES	1	2	
Crash Frequency/Year	0.29	0.89	+ 211.11
Crash Rate/mev	0.06	0.18	+ 178.42
INJURY CRASHES	10	5	
Crash Frequency/Year	2.86	2.22	-22.22
Crash Rate/mev	0.64	0.44	-30.40
PDO CRASHES	4	4	
Crash Frequency/Year	1.14	1.78	+ 55.56
Crash Rate/mev	0.25	0.35	+ 39.21
RIGHT-ANGLE/BROADSIDE CRASHES	13	9	
Crash Frequency/Year	3.71	4.00	+ 7.69
Crash Rate/mev	0.83	0.80	-3.63
Near-Side Right-Angle	6	6	
Crash Frequency/Year	1.71	2.67	+ 55.56
Crash Rate/mev	0.38	0.53	+ 39.21
Near-Side Right-Angle (SB & EB Traffic)	5	5	
Crash Frequency/Year	1.43	2.22	+ 55.56
Crash Rate/mev	0.60	0.84	+ 41.02
Near-Side Right-Angle (NB & WB Traffic)	1	1	
Crash Frequency/Year	0.29	0.44	+ 55.56
Crash Rate/mev	0.14	0.19	+ 37.20
Far-Side Right-Angle	7	3	
Crash Frequency/Year	2.00	1.33	-33.33
Crash Rate/mev	0.45	0.27	-40.34
LEFT-TURN LEAVING	2	0	
Crash Frequency/Year	0.57	0	-100
Crash Rate/mev	0.13	0	-100
REAR-END	0	2	
Crash Frequency/Year	0	0.89	+ Undefined
Crash Rate/mev	0	0.18	+ Undefined

* AADT values for US-61 and Hershey Road are estimated from the *Iowa Department of Transportation Traffic Volume Maps by City* (*87*). The before period uses 2002 values and the after period uses an average of the 2002 and 2006 values.

** Maze et al. (*2*) SPF in Table 3 was used to compute these expected values.

*** No statistical comparison was performed (< 3 years of after data).

periods, five of the six near-side right-angle crashes involved southbound traffic on US-61 colliding with eastbound traffic on Hershey Road. This distribution can possibly be explained by the fact that southbound traffic on US-61 is rounding a horizontal curve and coming down a relatively steep grade as it approaches Hershey Road. These alignment issues could be causing eastbound drivers on Hershey Road to have problems seeing and/or judging the speed of southbound traffic on US-61, regardless of the presence of the offset right-turn lane. The view of an eastbound driver on Hershey Road can be seen in the top portion of Figure 83. These alignment issues may explain why the southbound offset right-turn lane was not beneficial; however, they do not explain why the northbound offset appears to have been ineffective as well.

Another issue at this intersection which may explain why neither of the offset right-turn lanes appear to have been effective is that the median width is very narrow (14 to 16 feet). This geometry does not allow a minor road passenger car to be fully stored within the median; therefore, minor road drivers are forced to make a one-stage crossing or left-turn maneuver. As a result, the crossing/left-turning task for the minor road driver becomes increasingly complex as they must simultaneously search for an acceptable gap in expressway traffic coming from both the left and the right. This may lead to the near-side crash problem in that drivers may look left, then right, and then proceed without looking back to the left.

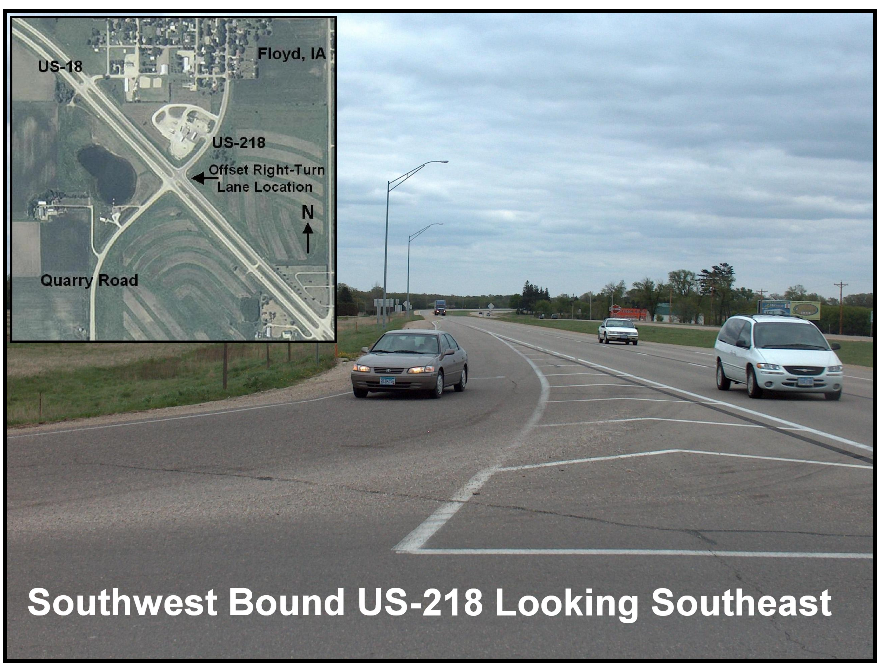


FIGURE 84. Offset right-turn lane at West Junction of US-18 & US-218 – Floyd, IA

The second example of an offset right-turn lane installation at a TWSC rural expressway intersection in Iowa was found at the West Junction of US-18 and US-218 just to the south of Floyd, IA. An aerial photo of this intersection is shown within Figure 84. In this area, US-18 was originally built to expressway standards sometime during the 1990's. The construction of the West US-18/US-218 intersection that resulted from this project included a conventional right-turn lane for northwest bound traffic on US-18 turning right onto US-218 toward Floyd. The intersection remained this way until late September of 2003 when IA-DOT District 2 converted this conventional right-turn lane into an offset right-turn lane as pictured in Figure 84.

This offset right-turn lane was installed due to a heavy volume of truck traffic exiting US-18 to access the truck stop located in the north quadrant of the intersection. The offset right-turn lane was constructed with district maintenance funds and the intent was to keep the cost to a minimum; therefore, the offset right-turn lane was designed as a normal parallel right-turn lane which later flares out at a 30:1 taper in order to achieve the desired offset. During the design process, a minimum departure sight triangle was used in deciding how much the right-turn lane needed to be offset. However, during pavement marking, a decision was made in the field to extend the two foot wide paved shoulder on the mainline throughout the offset right-turn lane. As a result, the outer edge of the gore area was painted twelve feet from the striped right-turn lane edge-line and the offset distance was reduced from what the designers had initially intended. As these markings wore off over time, the district attempted to increase the offset distance (gore area) by positioning the right-turn lane closer to the edge of pavement (as shown in Figure 84). David Little, Assistant IA-DOT District 2 Engineer, stated, "The offset seems to have been an improvement, but the overall consensus is that the right-turn lane is still not offset far enough." Therefore, District 2 is currently working on a project which will offset this right-turn lane by three or four more feet. In conjunction with this project, the district plans to place rumble strips within the gore area to encourage right-turning drivers to position themselves fully within the offset right-turn lane. Another indirect means of increasing the offset at this location may include moving the stop bar and stop sign on southwest bound US-218 closer to the mainline. Currently, they are positioned too far back (as shown in Figure 85) and, as a result, minor road drivers stopped at the stop bar do not get the full sight-distance advantage provided by the offset right-turn lane.

FIGURE 85. Stop bar location on southwest bound US-218

Before and after crash data for this offset right-turn lane conversion were obtained from ITSDS and are shown in Table 31. In the three year before period (1/1/2000 through 12/31/2002) the intersection experienced a total of 8 intersection-related crashes (4 injury and 4 PDO), giving an overall annual crash frequency of 2.67 crashes per year which matches the expected crash frequency based on the entering traffic volumes (*2*). The before period crash rate was 0.83 crashes per mev.

In the three year after period (1/1/2004 through 12/31/2006), there were also 8 intersection-related crashes (6 injury and 2 PDO), resulting in an annual crash frequency of 2.67 crashes per year; thus matching the before period and the expected crash frequencies. However, in the after period, the overall crash rate decreased by approximately six percent to 0.78 crashes per mev. Therefore, on the surface, it appears that the offset right-turn lane installation at this location did little to enhance safety. However, a further examination of near-side right-angle collisions (the crash type targeted by the treatment) does show improvement.

TABLE 31. Offset Right-Turn Lane Before-After Data Comparison (US-18 & US-218)

	BEFORE	AFTER	% CHANGE	SIGNIFICANT DIFFERENCE AT
ESTIMATED ENTERING AADT (US-18) *	7,350	8,000	+ 8.84	
ESTIMATED ENTERING AADT (US-218) *	1,425	1,350	-5.26	
ESTIMATED TOTAL ENTERING AADT	8,775	9,350	+ 6.55	
ESTIMATED ENTERING AADT (SW & NW) *	5,250	5,410	+ 3.05	
EXPECTED CRASH FREQUENCY/YEAR **	2.69	2.69	+ 0.10	
YEARS	3	3		
TOTAL CRASHES	8	8		
Crash Frequency/Year	2.67	2.67	0	$\alpha = 0.5000$
Crash Rate/mev	0.83	0.78	-6.15	
FATAL CRASHES	0	0	0	Not Valid (variances =0)
INJURY CRASHES	4	6		
Crash Frequency/Year	1.33	2.00	+ 50.00	$\alpha = 0.3174$
Crash Rate/mev	0.42	0.59	+ 40.78	
PDO CRASHES	4	2		
Crash Frequency/Year	1.33	0.67	-50.00	$\alpha = 0.2895$
Crash Rate/mev	0.42	0.20	-53.07	
RIGHT-ANGLE/BROADSIDE CRASHES	6	2		
Crash Frequency/Year	2.00	0.67	-66.67	$\alpha = 0.1667$
Crash Rate/mev	0.62	0.20	-68.72	
Near-Side Right-Angle	4	2		
Crash Frequency/Year	1.33	0.67	-50.00	$\alpha = 0.1151$
Crash Rate/mev (Total)	0.42	0.20	-53.07	
Crash Rate/mev (SWB & NWB Traffic)	0.70	0.34	-51.48	
Far-Side Right-Angle	2	0		
Crash Frequency/Year	0.67	0	-100	$\alpha = 0.2113$
Crash Rate/mev	0.21	0	-100	
RIGHT-TURN LEAVING	0	5		
Crash Frequency/Year	0	1.67	+ Undefined	$\alpha = 0.0189$ ***
Crash Rate/mev	0	0.49	+ Undefined	
LEFT-TURN LEAVING	1	0		
Crash Frequency/Year	0.33	0	-100	$\alpha = 0.2113$
Crash Rate/mev	0.10	0	-100	
REAR-END	1	0		
Crash Frequency/Year	0.33	0	-100	$\alpha = 0.2113$
Crash Rate/mev	0.10	0	-100	
SIDESWIPE (SAME DIRECTION)	0	1		
Crash Frequency/Year	0	0.33	+ Undefined	$\alpha = 0.2113$
Crash Rate/mev	0	0.10	+ Undefined	

* AADT values for US-18 and US-218 are estimated from the Iowa DOT Traffic Volume Maps by City (*87*). The before period uses 2001 values and the after period uses 2005 values.

** Maze et al. (*2*) SPF in Table 3 was used to compute these expected values.

*** Statistically significant difference in sample means assuming unequal variances at a 90% level of confidence using a one-tailed t-test.

In the before period, the intersection experienced a total of six right-angle collisions. Of these, four were near-side collisions (all involving vehicles on southwest bound US-218 colliding with vehicles on northwest bound US-18, which is the approach where the offset right-turn lane was eventually installed); thus giving a near-side right-angle crash frequency of 1.33 crashes per year and a corresponding near-side right-angle crash rate of 0.42 crashes per mev. In the after period, only two near-side right-angle crashes occurred (both involving vehicles on southwest US-218 and northwest US-18); thus reducing the annual near-side right-angle crash frequency to 0.67 crashes per year (a 50 percent reduction) and reducing the near-side right-angle crash rate to 0.20 crashes per mev (a 53 percent reduction). Therefore, according to this naïve before-after comparison, it appears that the offset right-turn lane at this location has been a safety improvement in terms of reducing near-side right-angle collisions as the decrease was statistically significant with 88 percent confidence ($\alpha = 0.12$).

However, it is interesting to note that in the after period, there was a statistically significant increase in "right-turn leaving" collisions involving a right-turning vehicle on northwest US-18 which used the offset right-turn lane, turned at a high rate of speed, lost control, slid through the throat of the intersection, and collided with a vehicle on southwest US-218 which was stopped at the stop sign waiting to enter the intersection. A review of the official crash reports revealed that two of these crashes occurred under foggy conditions and another occurred when a motorcycle lost control on loose gravel. This crash type may be an indication that drivers are interpreting the tapered offset right-turn lane design used at this location as a high-speed right-turn exit ramp which is consequently encouraging drivers to make the right-turn at a higher rate of speed than is safe for the conditions. A driver in one of the crash reports actually mentioned that, because of the fog, he thought the offset right-turn lane was a ramp onto nearby I-35. Some possible fixes to prevent this crash type from occurring include: 1) paving the shoulder adjacent to the offset right-turn lane to keep excess gravel out of the turning lane, 2) increasing the turning radius for the exiting offset right-turn lane so that it is more like an exit ramp, 3) using a parallel offset right-turn lane design (see Figure 80B) as opposed to the tapered type design used by the IA-DOT (see Figures 81 and 84) so that the offset right-turn lane does not appear to be an exit ramp, 4) posting an advisory speed plaque with the message "RIGHT-TURN XX MPH" along the deceleration lane far enough in advance so that the exiting right-turn driver can make a safe slowing and turning maneuver, and/or 5) placing a divisional island on the minor road approach to help shield minor road traffic.

3.8.3.2 Nebraska Experience

A third example of an offset right-turn lane installation at a TWSC rural expressway intersection was found a few miles to the southeast of Lincoln, Nebraska at the intersection of Nebraska Highway 2 (N-2) and 148th Street. N-2 was converted from a two-lane undivided highway into an expressway in late 1997. The initial N-2/148th Street intersection that resulted from this project did not provide any right-turn lanes for traffic exiting N-2. 148th Street is a two-lane undivided paved county road which essentially functions as a bypass on the east edge of Lincoln. In late 1998, an NDOR traffic engineering study identified the need to install a right-turn lane on westbound N-2 for traffic turning northward onto 148th Street. This study indicated that: 1) current right-turn traffic volumes at the intersection met NCHRP 279 (*88*) volume warrants for a full-width right-turn lane, 2) westbound right-turning traffic often used the paved shoulder to complete the turn, 3) a heavy volume of truck traffic was using 148th Street, and 4) although ISD was adequate, the intersection is placed on a crest vertical curve such that westbound traffic on N-2 does not see the intersection until just over the crest as shown in the upper left of Figure 86. As a result of these observations, a decision was made to construct an offset right-turn lane. The parallel offset right-turn lane shown in Figure 86 was constructed around July of 2003 (the exact construction dates could not be determined). NDOR personnel estimated that the offset distance is twelve feet. In addition, the same project also constructed a divisional (splitter) island on southbound 148th Street and installed an additional stop sign there as shown in the lower left of Figure 86.

Crash data for this intersection were obtained from NDOR and is summarized in Table 32. In the 5.50 year before period (1/1/1998 through 6/30/2003) there were a total of three reported PDO crashes which occurred at the intersection, giving an average annual crash frequency of 0.55 crashes per year (recall that the offset right-turn lane installation at this location was based on a volume warrant, not poor safety performance). In the 2.50 year after period (7/1/2003 to 12/31/2005) there were a total of five intersection-related collisions (1 fatal and 4 PDO), giving an average annual crash frequency of 2.0 crashes per year. Therefore, the crash frequency at this intersection increased by approximately 267 percent

FIGURE 86. Offset right-turn lane at N-2 & 148th Street – near Lincoln, NE

TABLE 32. Offset Right-Turn Lane Before-After Data (N-2 & 148th Street)

	BEFORE	AFTER	% CHANGE *
YEARS	5.50	2.50	
TOTAL INTERSECTION-RELATED CRASHES	3	5	
Crash Frequency/Year	0.55	2.00	+ 266.67
FATAL CRASHES	0	1	
Crash Frequency/Year	0	0.40	+ Undefined
INJURY CRASHES	0	0	
Crash Frequency/Year	0	0	0
PDO CRASHES	3	4	
Crash Frequency/Year	0.55	1.60	+ 193.33
RIGHT-ANGLE/BROADSIDE CRASHES	2	1	
Crash Frequency/Year	0.36	0.40	+ 10.00
Near-Side Right-Angle	1	0	
Crash Frequency/Year	0.18	0	-100
Far-Side Right-Angle	1	1	
Crash Frequency/Year	0.18	0.40	+ 120.00
REAR-END CRASHES	1	3	
Crash Frequency/Year	0.18	1.20	+ 560.00
OTHER CRASHES	0	1	
Crash Frequency/Year	0	0.40	+ Undefined

* No statistical comparison was performed (< 3 years of after data).

after the offset right-turn lane was installed; however, further examination of near-side right-angle crashes shows more positive results.

Of the three crashes which occurred during the before period, only one was a near-side right-angle collision. This crash did involve a vehicle on southbound 148th Street colliding with a westbound vehicle on N-2 (the approach where the offset right-turn lane was eventually installed); giving a near-side right-angle crash frequency of 0.18 crashes per year. It was noted in the crash report that the southbound driver's sight distance was obstructed by an uninvolved vehicle turning right from the paved shoulder of westbound N-2; therefore, this collision may have been prevented had the offset right-turn lane been in place at that time. In the after period, even though the overall crash frequency dramatically increased, no near-side right-angle crashes occurred at the intersection, giving a 100 percent reduction for this crash type. Therefore, it appears that the offset right-turn lane was a safety improvement in terms of preventing near-side right-angle collisions. However, it should be mentioned that the collision classified as "other" in the after period was a single vehicle, run-off-road, PDO crash under daylight and dry conditions in which a westbound vehicle on N-2 took evasive action to prevent a near-side right-angle collision with a southbound vehicle on 148th Street which had pulled out in front of him. It was not stated whether or not a right-turning vehicle was present at the time of this collision; however, if there were, the offset right-turn lane may have helped prevent a more severe crash.

3.8.4 Offset Right-Turn Lane Summary

The assumed safety benefit of offset right-turn lanes is that they eliminate the sight distance obstruction created by the presence of right-turning expressway vehicles positioned in a conventional right-turn lane, thereby allowing minor road drivers to make better gap acceptance decisions when entering the near-side intersection. Expressway intersections most likely to benefit from offset right-turn lanes include: 1) intersections with a history of near-side right-angle collisions resulting from right-turning expressway vehicles obstructing minor road driver sight lines, and 2) intersections with large right-turn volumes (especially trucks) leaving the expressway, in combination with large volumes of minor road and expressway traffic on the corresponding adjacent approaches. No volume warrants for their use have been developed, but Zeidan and McCoy (*86*) stated that their need depends on the probability of the sight-distance problem occurring (i.e., traffic is present in the right-turn lane, on the minor road, and on the expressway) and its duration which is a function of traffic volumes and their arrival distributions. A research project examining the conditions warranting offset right-turn lanes is currently being conducted at the University of Nebraska-Lincoln (*89*).

TABLE 33. Offset Right-Turn Lane Safety Effectiveness Summary

	US-61 & Hershey Road No RTL → Offset % Change *	US-18 & US-218 Conventional RTL → Offset % Change **	N-2 & 148th St. No RTL → Offset % Change *
Total Crash Frequency/yr	+14	0	+267
Total Crash Rate/mev	+2	-6	
Right-Angle Frequency/yr	+8	-67	+10
Right-Angle Crash Rate/mev	-4	-69	
Near-Side Right-Angle Frequency/yr	+56	-50	-100
Near-Side Right-Angle Crash Rate/mev	+39	-53	

** No statistical evaluation was performed (< 3 years of after data)*
*** Statistical evaluation conducted, but none of the values reported here were statistically significant changes at a 90% level of confidence using a one tailed t-test*

Two of the three before-after offset right-turn lane case studies presented in this thesis revealed a reduction in the annual frequency of near-side right-angle collisions. Table 33 summarizes these results; however, only the US-18/US-218 intersection near Floyd, IA demonstrated the effects of offsetting a conventional right-turn lane as no right-turn lanes previously existed at the other two locations. Given the limited number of sites, the fact that there were less than three years of after data at two of the three sites, and the limitations of the naïve before-after analysis methodology, no definitive conclusions regarding the safety effects of offset right-turn lanes at TWSC rural expressway intersections can be drawn from this study. Further evaluation with more sites and more data is clearly necessary. Moreover, it must be stated that offset right-turn lanes are only meant to enhance sight distance and reduce the possibility of near-side right-angle collisions when right-turning vehicles are present on the expressway. By only examining the crash data, there is no way of knowing how often the sight-distance problem occurred in the before period or how often it would have occurred in the after period. It is also hard to know whether or not right-turning vehicles were present at the time of the reported near-side collisions unless specifically stated in the official crash reports. Sometimes an officer or the drivers involved may note the presence of an uninvolved vehicle, but sometimes not; consequently, this information is sketchy at best and can be time consuming to obtain. Therefore, a better means of evaluating the effectiveness of the offset right-turn lane treatment in the future may be to conduct an observational before-after conflict analysis.

Even though no conclusions regarding the safety effects of offset right-turn lanes could be drawn from this study, the theory behind this countermeasure is sound; thus offset right-turn lane design guidance should be incorporated into the AASHTO Green Book (*3*) as no guidance on this strategy is currently available there. The lessons learned from these case studies are important in this regard. The most important design aspect of an offset right-turn lane is that it should provide the minor road driver with a clear departure sight triangle to the left (i.e., sufficient sight distance along the near-side expressway lanes) when right-turning vehicles are present on the expressway. The required right-turn offset distance may vary from intersection to intersection based on each intersection's unique geometry (skew, horizontal curvature, approach grades, design speed, stop bar placement, etc.); therefore, intersection design plans should be checked to ensure that adequate ISD is provided. In addition, the following design guidance should be considered: 1) to ensure offset right-turn lanes are utilized properly, rumble strips could be placed in the gore area with the edge lines painted through the rumble strips to get a vertically painted edge line face, 2) to indirectly increase the offset distance, the stop bar/sign on the minor road approach could be moved as close to the mainline as safely possible or the outside expressway lane line edge could be dashed through the intersection to encourage minor road drivers to stop a little closer to the mainline, and 3) additional precautions should be taken to prevent right-turn leaving collisions such as ensuring the offset right-turn lane does not appear to be an exit ramp (i.e., constructing parallel rather than tapered offset right-turn lanes), posting advisory speed signs in advance of or along the offset right-turn lane, constructing a splitter island on the minor road approach, and/or paving the shoulder adjacent to the offset right-turn lane throughout its turn radius.

3.9 OFFSET LEFT-TURN LANES CASE STUDY

3.9.1 Description

The purpose of providing exclusive left-turn lanes within the median on expressway intersection approaches is to provide space for deceleration and storage of left-turning vehicles (*3*). By removing left-turning traffic from the high-speed through lanes, speed differentials in the through lanes are reduced, thus enabling through traffic to pass by with little conflict or delay and improving the overall safety and capacity of the intersection. On page 689, the AASHTO Green Book (*3*) states, "Deceleration lanes are always advantageous, particularly on high-speed roads, because the driver of a vehicle leaving the highway has no choice but to slow down in the through traffic lane if a deceleration lane is not provided." On page 716, the Green Book goes on to say, "Inefficiencies in operations may be evident on divided highways where such lanes are not provided. Median left-turn lanes, therefore, should be provided at intersections and at other median openings where there is a high volume of left turns or where vehicular speeds are high."

The limited research found assessing the safety effects of providing left-turn lanes at rural expressway intersections has supported these statements by showing that their presence does indeed improve safety. A crash model developed by Harwood et al. (*9*) in 1995 using 153 TWSC divided highway intersections in rural California showed that the presence of exclusive left-turn lanes on the divided highway significantly reduced the total multi-vehicle intersection crash frequency. However, a potential problem with installing conventional median left-turn lanes at divided highway intersections is that opposing left-turn vehicles exiting a divided highway can obstruct each other's line-of-sight and hinder each other's ability to see oncoming expressway traffic through which they must turn. This scenario is illustrated in Figure 87A. Previous research has indicated that such sight distance restrictions can lead to collisions between left-turning vehicles exiting the expressway and opposing through traffic (i.e., "left-turn leaving" collisions) (*90*).

The most common solution to this intersection sight distance problem is to offset the left-turn lanes. In this design, the left-turn lanes are shifted laterally to the left within the median as far as necessary so that opposing left-turn vehicles no longer obstruct each other's line-of-sight. This design concept is illustrated in Figure 87B. Two different types of offset

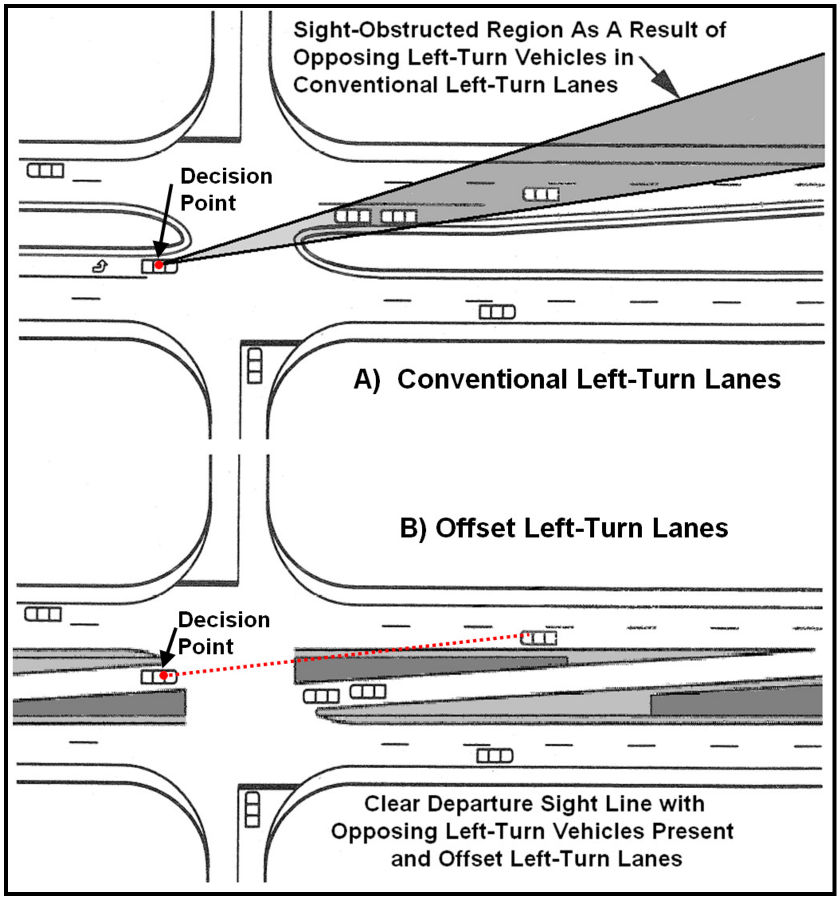


FIGURE 87. Offset left-turn lane design concept illustration

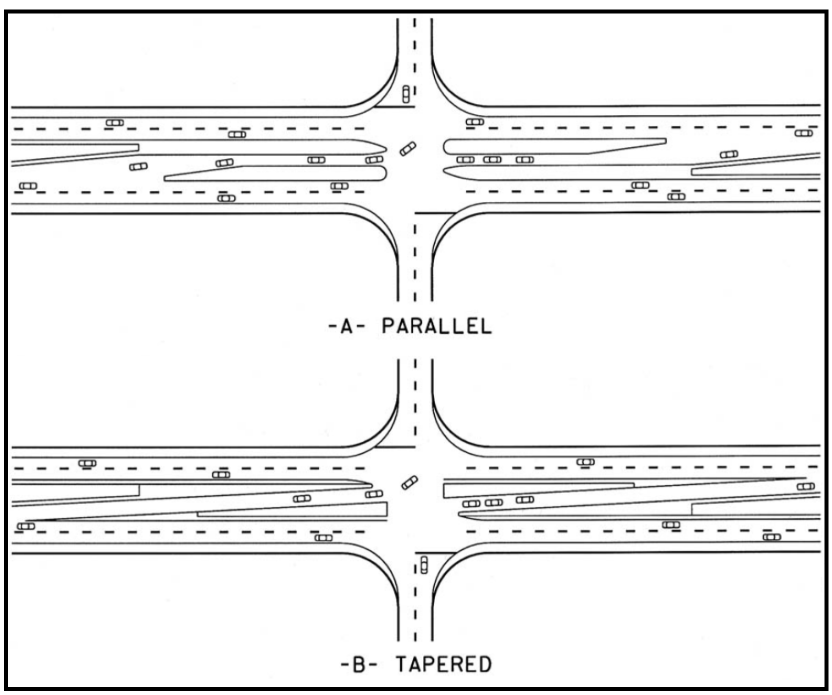


FIGURE 88. Green Book Exhibit 9-98; parallel and tapered offset left-turn lanes (3)

left-turn lane designs (parallel and tapered) are presented in Green Book (*3*) Exhibit 9-98 (Figure 88). Offset left-turn lanes are expected to improve safety at TWSC rural expressway intersections by enhancing visibility for left-turning drivers leaving the expressway when opposing left-turn vehicles are present; thereby allowing them to make better decisions when selecting gaps in the opposing expressway traffic stream. As such, they are expected to reduce left-turn leaving type collisions, as well as rear-end crashes between through vehicles on the opposing approach (*16*). Additional advantages of offset left-turn lanes include: 1) reduced potential for conflict between opposing left-turn movements within the median (i.e., the design allows simultaneous left-turns), 2) increased left-turn capacity, 3) side-by-side queuing is limited by storing mainline left-turn traffic separate from minor road traffic using the median, 4) left-turn traffic is stored further away from the adjacent mainline high-speed through traffic, 5) the design is adaptable to a wide range of median widths, and 6) implementation can usually be accomplished without acquiring additional right-of-way (*3, 6*).

Although Harwood et al. (*9*) observed no operational problems at three signalized rural expressway intersections with offset left-turn lanes (two tapered, one parallel), and Schurr et al. (*25*) roughly estimated the safety benefits of providing offset left-turn lanes at TWSC rural expressway intersections, no reliable estimates of their safety effectiveness have been scientifically determined through a rigorous before-after safety evaluation (*16*). It is believed that offset left-turn lanes will improve safety and a negative binomial model developed by Khattak et al. (*23*) showed that, in Nebraska, expressway approaches with offset left-turn lanes do indeed have fewer crashes than approaches with conventional or no left-turn lanes; however, some potential pitfalls of their installation may still exist. For instance, Schurr et al. (*25*) found that offset left-turn lanes seem to encourage left-turning drivers to slow down more in the passing lane of the expressway prior to entering the bay than conventional left-turn lanes do. The larger speed differentials on the mainline created by this behavior could lead to an increase in rear-end crashes as compared with conventional left-turn lane designs. This finding may be evidence that, with offset left-turn lanes, mainline left-turning drivers must enter the median sooner than expected (*6*). It may also reflect the lack of driver familiarity with the offset left-turn lane design (*9*).

Due to the unusual nature of the design, unfamiliar and elderly drivers may be confused by the change in traffic patterns and the unclear right-of-way regulations which may exist within the median (*6, 16*). In addition, offset left-turn lanes may increase the potential for wrong-way entry by both minor road and opposing through traffic (*9, 70*). There is also some concern that drivers would not use the lanes as intended since the small island to the right of the offset lane is typically flush and painted in rural applications (*27*). Figure 89 is evidence that this concern is valid. If left turning drivers do not respect the intended channelization, the safety benefits this design intends to offer will be negated; therefore, public information and education campaigns should be considered when such treatments are used for the first time in a given area (*16*). Other potential disadvantages of offset left-turn lanes include: 1) increased difficulty in making U-turns, 2) increased difficulty of snow removal and deicing activities on the separate left-turn roadways, 3) increased drainage requirements, and 4) challenging to install in conjunction with MALs (*6, 9*).

FIGURE 89. Observed misuse of offset left-turn lane in Nebraska

3.9.2 Existing Design Guidance

The AASHTO Green Book (*3*) provides some general design guidance for offset left-turn lanes in Chapter 9 on page 723 with Green Book Exhibit 9-98 (Figure 88) illustrating the parallel and tapered type designs. On page 723, the Green Book states:

For medians wider than about 18 feet, it is desirable to offset the left-turn lanes so that it will reduce the width of the divider to 6 to 8 feet immediately before the intersection, rather than to align them exactly parallel with and adjacent to the through lane. This alignment will place the vehicle waiting to make the turn as far to the left as practical, maximizing the offset between the opposing left-turn lanes, and thus providing improved visibility of opposing through traffic.

Although the Green Book states that moving the left-turning vehicles as far to the left as practical maximizes the offset between opposing left-turn lanes, it does not provide any guidance on the minimum offset required to provide adequate sight distance. The minimum amount of sight distance necessary for a left turn maneuver from a major road is described as ISD Case F on pages 674 through 676 of the Green Book (*3*). In 1992, McCoy et al. (*90*) conducted a study to develop guidelines for offsetting opposing left-turn lanes at right-angle intersections on level, tangent sections of four-lane divided roadways. The development of these guidelines began with a study of left-turn vehicle positioning in 12 foot wide left-turn lanes within 16 foot wide curbed medians having four foot wide medial separators. This study defined the left-turn offset distance as “The lateral distance between the left edge of a left-turn lane and the right edge of the opposing left-turn lane.” If the right edge of the opposing left-turn lane is to the left of the left edge of the left-turn lane, the offset is defined as “negative”; if it is to the right, the offset is defined as “positive”. An illustration of negative and positive offsets is shown in Figure 90. These definitions could be included in the AASHTO Green Book (*3*).

As a result of this study, McCoy et al. (*90*) determined that the minimum required offset is always positive, indicating that offset left-turn lanes must always be positively offset in order to be effective. Under high-speed conditions ($\geq$ 50 mph), the minimum offset was determined to be 1.5 feet when the opposing left-turn vehicle is a passenger car and 3.0 feet when the opposing left-turn vehicle is a truck. However, desirable offsets (which provide the opposing left-turn vehicles with unrestricted sight distance) were determined to be 2.0 and 3.5 feet in these same situations, respectively. The trigonometry used to derive these values is illustrated in Figure 91. These values are only applicable for right-angle intersections on level, tangent sections of four-lane divided roadways with 12 foot lanes. In addition, the required sight distance used to determine these minimum offsets was computed using ISD

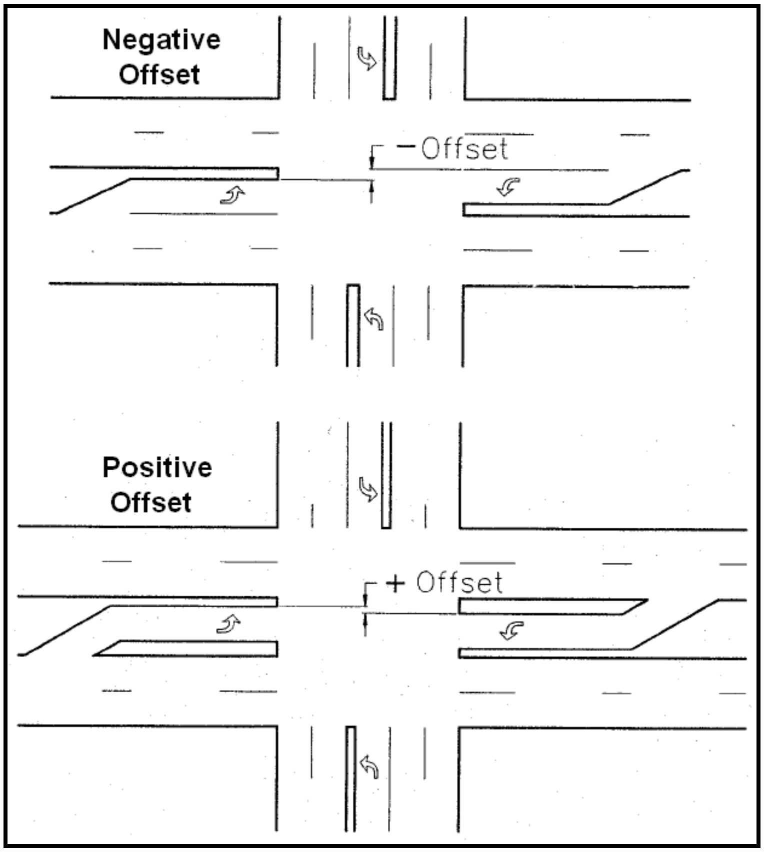


FIGURE 90. Illustration of negative and positive left-turn offsets (90)

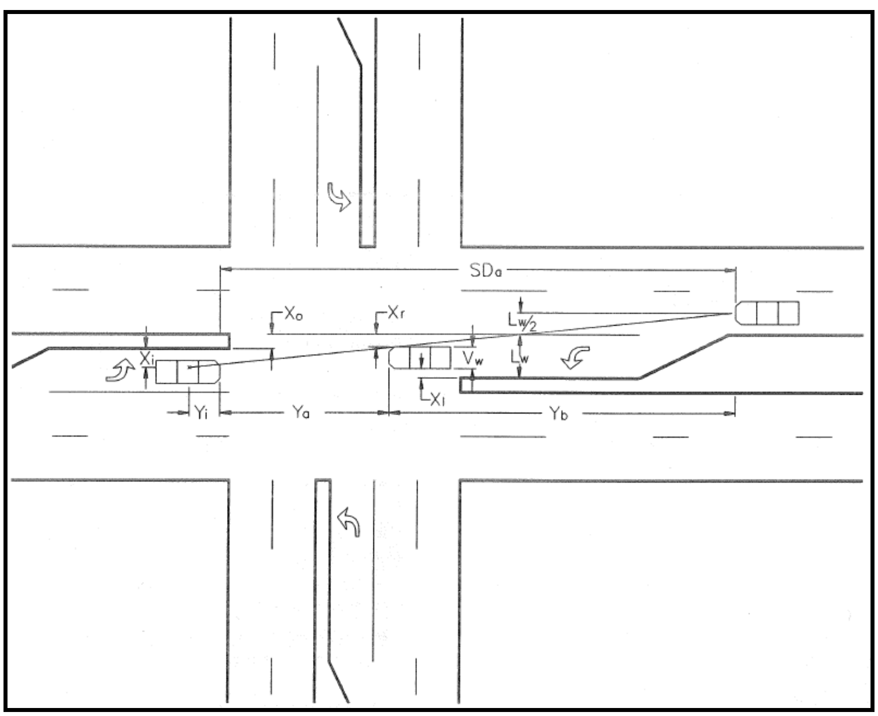
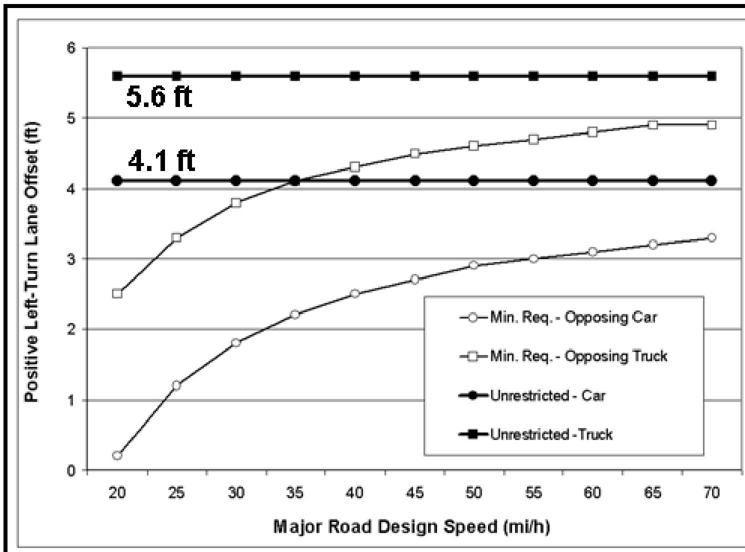


FIGURE 91. Minimum left-turn lane offset calculation trigonometry (90)

standards from the 1990 edition of the AASHTO Green Book which have since been redefined using a gap acceptance model.

In 2001, Staplin et al. (*38*) developed the graph shown in Figure 92 to update the minimum left-turn offset distances determined by McCoy et al. (*90*) for accommodation of older drivers. A more detailed description of the development of this graph is given in Staplin et al. (*70*). The values in the graph were determined using the gap acceptance model for ISD Case F found in the 2001 edition of the Green Book which is the same standard found in the current edition (*3*); however, the values in Figure 92 are conservative to accommodate elderly drivers. Staplin et al. (*38*) recommended that the unrestricted sight distance offsets in Figure 92 be used whenever possible, thus providing a margin of safety for elderly drivers. They also recommend using the offsets which assume the opposing left-turn vehicle is a truck at intersections where there is a high probability of left-turning trucks. Of course, the required offset distances will vary from intersection to intersection depending on each intersection's unique geometry (horizontal curvature, approach grades, design speeds, etc.); therefore, intersection design plans should still be checked to ensure that left-turning drivers leaving the expressway are provided a clear departure sight triangle (i.e., sufficient sight distance along the opposing expressway lanes) while opposing left-turn vehicles are present.



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.

FIGURE 92. Minimum and unrestricted left-turn offset distances (38)

MUTCD (*31*) Figure 2B-15 (Figure 93) presents a signing and marking plan for TWSC divided highway intersections with medians less than 30 feet wide and parallel offset left-turn lanes. However, the figure illustrates negatively offset left-turn lanes which do not provide enough ISD when opposing left-turn vehicles are present as established in previous research (*38, 90*). This figure may explain why some STAs are inappropriately constructing offset left-turn lanes in this manner as shown in Figure 94. Therefore, MUTCD Figure 2B-15 should be modified to illustrate positively offset left-turn lanes and the term "offset" should be used in the title of the figure rather than the word "separated" in order to stay consistent with the terminology used in the AASHTO Green Book (*3*). Staplin et al. (*38*) also recommended the signing and marking treatments shown in Figure 95 at TWSC divided highway intersections with medians greater than 30 feet wide and offset left-turn lanes (tapered or parallel) to help reduce the potential for wrong-way entry onto the divided highway.

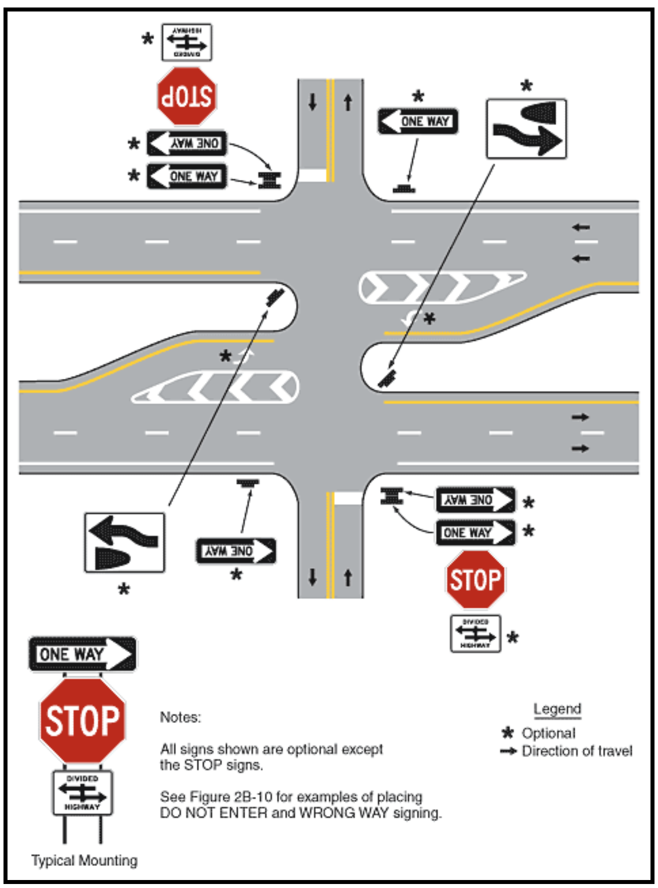


FIGURE 93. MUTCD Figure 2B-15; example of ONE WAY signing for divided highways with medians less than 30 feet and separated left-turn lanes (31)

FIGURE 94. Offset left-turn lanes designed with a negative offset (incorrect design)

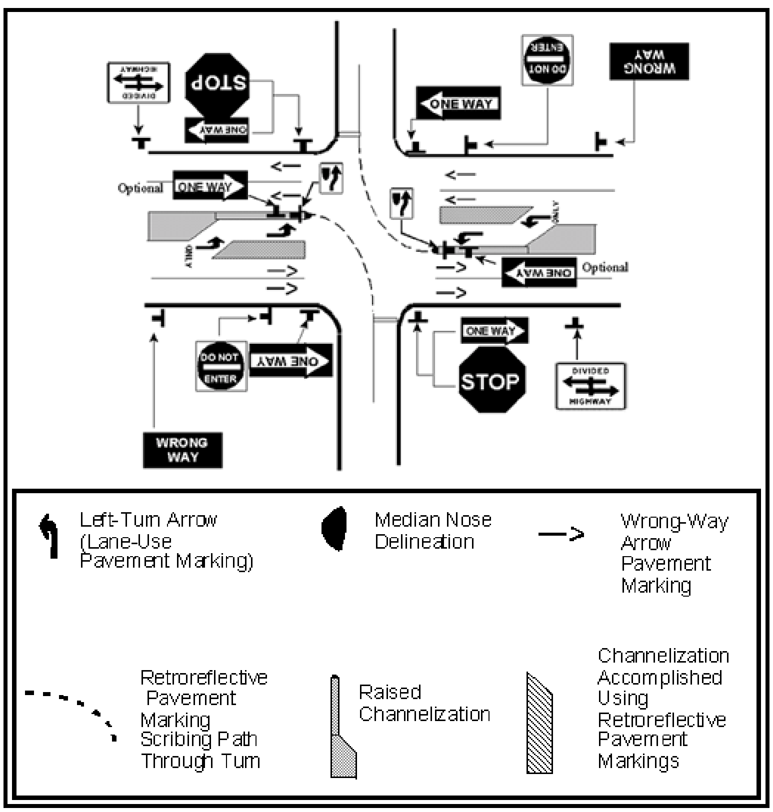


FIGURE 95. Recommended regulatory signing and marking for TWSC rural expressway intersection with a median width of 30 feet or greater and offset left-turn lanes (38)

As shown in Figure 88, there are two types of offset left-turn lanes, parallel and tapered. According to the Green Book (*3*), both designs provide similar advantages, with tapered offsets being the preferred design option for turning radii allowance where a large number of left-turning trucks with long rear overhangs are expected. In addition, the Green Book states that tapered offset left-turn lanes have been primarily used at signalized intersections and that they are normally constructed with a four foot nose between the left-turn lane and the opposing through lanes. On the other hand, the Green Book states that parallel offset left-turn lanes may be used at both signalized and unsignalized intersections. Based on the literature review, there is no evidence that one design is superior to the other; however, Bonneson et al. (*27*) stated that experience with the tapered configuration indicates that some sight distance obstruction can still be incurred when the tapered storage area contains several queued vehicles. On the contrary, when the parallel configuration is used, all queued left-turn vehicles are removed from the opposing left-turn driver's line-of-sight presuming the storage capacity of the left-turn lanes is not exceeded. NCHRP 375 (*9*) showed that both tapered and parallel designs are feasible when the median width is at least 26 feet; however, they can be constructed in narrower medians by reducing lane widths and/or medial separators, which result in less than desirable offsets between opposing left-turn lanes.

Because offset left-turn lanes are discussed within the AASHTO Green Book (*3*), their use as a crash countermeasure is much more prevalent than the use of offset right-turn lanes. In a 1995 survey of 44 STAs, NCHRP 375 (*9*) indicated that 62 percent had used offset left-turn lanes. In a more recent survey of 28 STAs conducted by Maze et al. (*2*), 79 percent stated that they have used or plan to use offset left-turn lanes at rural expressway intersections. As a result, many STAs have incorporated offset left-turn lanes into their design manuals as standard practice for intersection design on divided highways; however, each state seems to have their own unique design standards and warrants for their use. For instance, standard offset left-turn lane designs from Illinois, Iowa, and Nebraska are presented in Figure 96. Because each state's design standards and warrants are unique, "best practices" for design should be compiled and incorporated into the Green Book (*3*).

According to NCHRP 375 (*9*), the Illinois DOT had the most extensive experience with offset left-turn lanes as of 1995. Today, standard plans for both tapered (Figure 96) and

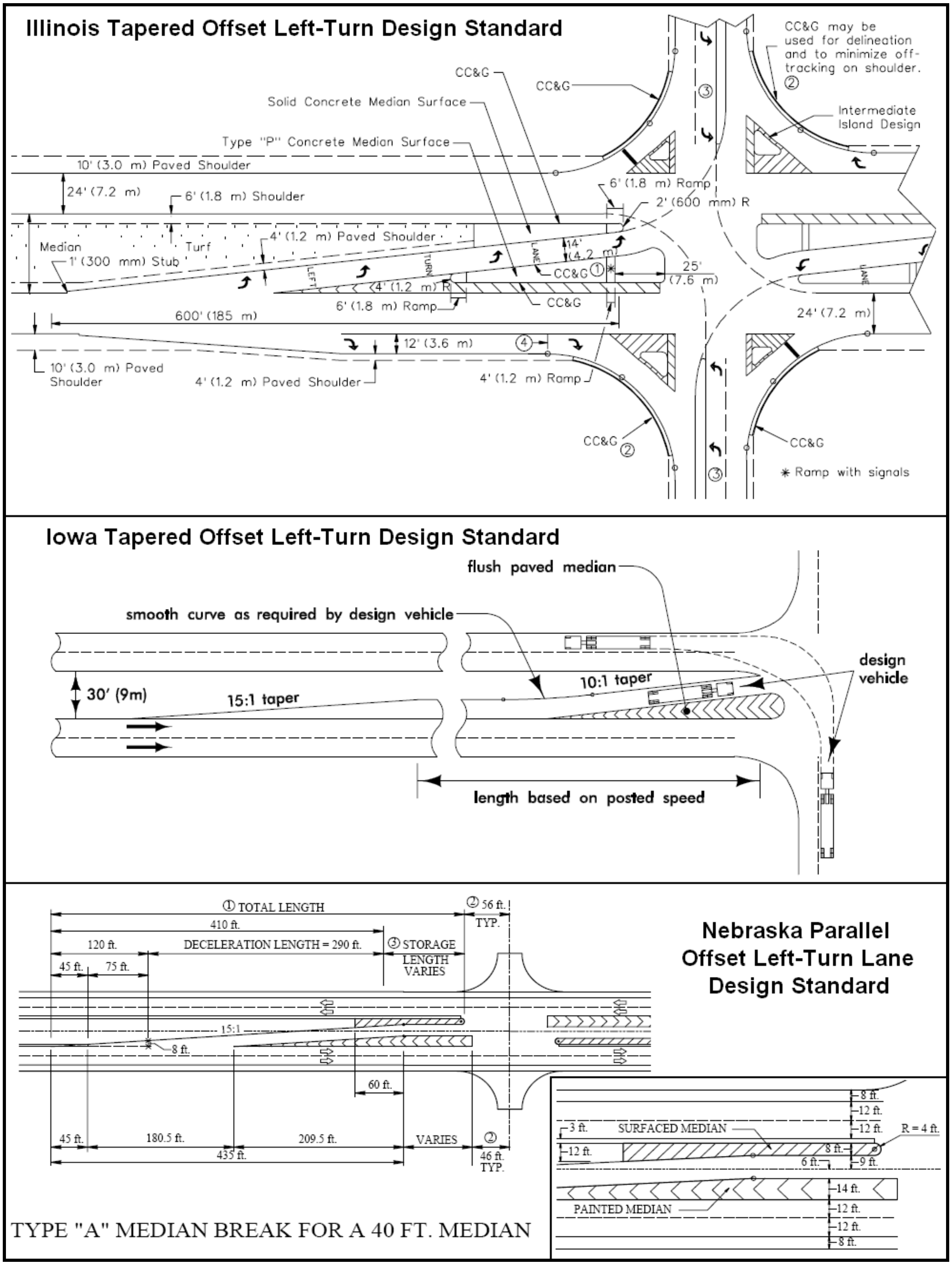


FIGURE 96. Offset left-turn lane designs in Illinois, Iowa, and Nebraska (7, 85, 91)

parallel offset left-turn lanes appear in Chapter 36, Section 3.03(c) of the Illinois' Bureau of Design and Environment Manual (*91*) which provides the following guidelines for their use:

> *Provide a tapered offset left-turn lane design where at least two of the following are applicable: 1) The median width is equal to or greater than 40 feet and only one left-turn lane in each direction on the mainline highway is required for capacity, 2) the current mainline ADT is 1500 or greater and the left-turn design hourly volume (DHV) in each direction from the mainline is greater than 60 vehicles per hour (vph) [Under these conditions, vehicles waiting in opposing left-turn lanes have the probability of obstructing each other's line of sight], and 3) the intersection will be signalized. Parallel offset left-turn lanes offer the same advantages as the tapered design; however, they may be used at intersections with medians less than 40 feet but greater than 13 feet.*

NCHRP 375 (*9*) also stated that the Illinois DOT has discounted most of the potential disadvantages of offset left-turn lanes discussed at the end of the "Description" section based on their operating experience. Illinois has found that driver confusion associated with offset left-turn lanes can be minimized through the use of proper signage and pavement markings (i.e., advance guide signing and pavement arrows on the entrance to the left-turn lane).

Iowa's Highway Design Manual (*85*) also contains design standards for tapered offset left-turn lanes as illustrated in Figure 96; however, they do not have a standard plan for parallel offset left-turn lanes. Chapter 6C-5 of Iowa's Highway Design Manual (*85*) states:

> *The use of offset (tapered) left-turn lanes should be limited on rural intersections. They should be considered only if traffic signals will likely be installed or if opposing left-turning vehicles create a significant sight distance problem. If offset left-turn lanes are used, the median width should be reduced to 30 feet.*

In Nebraska, a divided highway intersection with parallel offset left-turn lanes is considered a "Type A" median break (see Figure 96). The median width for this median type is 40 feet and the left-turn lanes are positively offset by three feet. In comparison, Nebraska's typical "Type B" median break (an intersection with conventional left-turn lanes in a 40 foot wide median) has a negative 25 foot offset. NDOR has also used tapered offset left-turn lanes at TWSC rural expressway intersections; however, their design manual does not currently contain any design standards for this type of median break. Chapter 4, Section 5.B.4(a) of the NDOR Roadway Design Manual (*7*) gives the following design guidance for parallel offset left-turn lanes:

> *Type A median breaks may be used at intersections of the mainline with paved public roads where there is high probability of turning vehicles blocking the opposing turning driver's view. A special traffic study will be required to justify the use of this type of intersection. The length for a Type A median break will consist of: 1) 120 feet of 15:1 taper to shift the turning traffic 8 feet from the through lane, 2) a deceleration lane length of 290 feet to slow traffic from 55 mph to a full stop (it is assumed that turning traffic will slow by 10 mph prior to entering the median break), 3) a minimum storage length of 50 feet providing storage for two cars at 25 feet per car or 100 feet if the percentage of trucks exceeds 10 percent, providing storage for one car at 25 feet and one truck at 75 feet.*

In 2003, Schurr et al. (*25*) compared driver behavior at TWSC rural expressway intersections in Nebraska with Type A and Type B median breaks and, as a result, made recommendations that NDOR revise their Type A median break design standards. These recommendations included: 1) flattening the offset left-turn lane entry taper to 20:1, 2) adding advance median signage announcing the presence of the approaching left-turn lane, and 3) changing the surfacing type/texture in the median areas between the offset left-turn lanes and the adjacent opposing direction through lanes. Figure 97 shows how different parallel and tapered offset left-turn lane designs have been created in Nebraska through a combination of tapered versus reverse-curve lane entry and surfaced versus turf medians. Figure 97 also demonstrates how the different designs can produce a drastic difference in approaching driver perception. The offset left-turn lane in the lower left corner is a tapered offset left-turn lane as illustrated in Figure 88B, while the other three are examples of parallel offset left-turn lanes as shown in Figure 88A. However, the two designs pictured on the left have tapered lane entry while the two designs on the right have reverse-curve lane entry. The two designs pictured on the top have turf medians all the way through the median nose, while the two designs on the bottom have surfaced medians near the intersection. Notice how the tapered entry with turf surfacing pictured in the upper left corner seems to provide an approaching left-turn driver with a better target (i.e., a better sense of where the intersection is ultimately located). In addition, the turf median would likely provide better visual delineation for opposing through traffic than the surfaced median as shown in Figure 98.

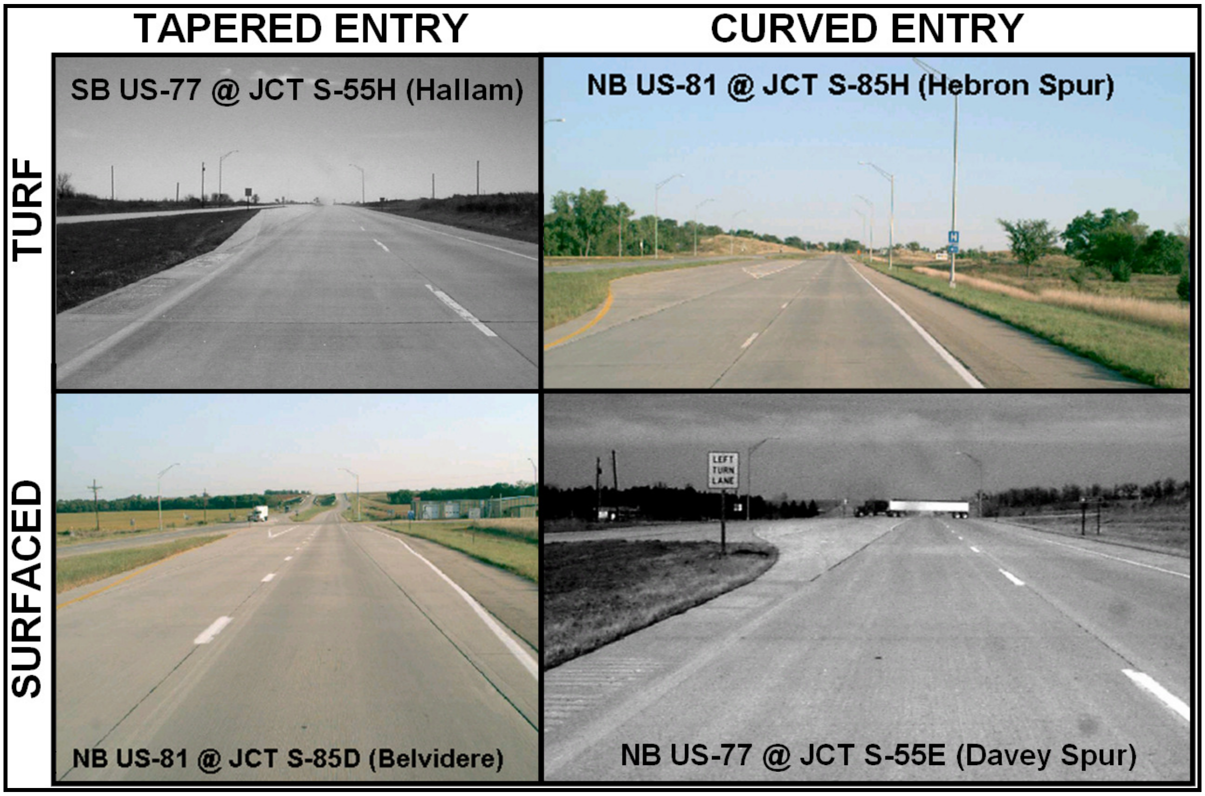


FIGURE 97. Different offset left-turn lane design applications in Nebraska

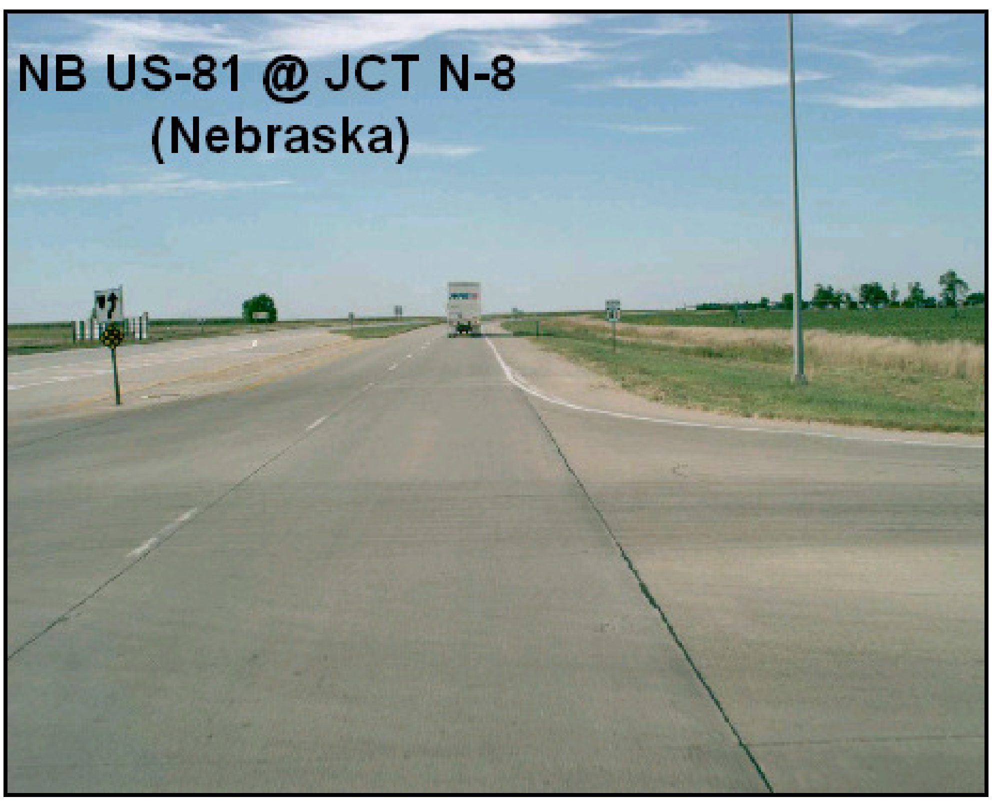


FIGURE 98. Opposing through driver's perspective of offset left with surfaced median

3.9.3 Case Studies of Implementation & Safety Effectiveness

3.9.3.1 North Carolina Experience

For the purpose of this case study, the NCDOT Safety Evaluation Group (*58*), a subdivision of their Traffic Safety Systems Management Section, conducted safety evaluations at two high-speed TWSC expressway intersections where offset left-turn lanes had been installed. These before-after spot safety evaluations were at the intersections of: 1) US-421 (Carolina Beach Road) and SR-1576/1531 (River Road/South Seabreeze Road), and 2) US-421 and SR-1524 (Golden Road). Each evaluation is briefly summarized here; however, further details can be found in the original reports (*92, 93*). The before and after crash data given in these reports were compared in terms of percent change; however, no analyses were conducted in the original reports to determine if the changes were statistically significant. Therefore, additional statistical comparisons were conducted here.

The first safety evaluation conducted by the NCDOT was at the intersection of US-421 and SR-1576/1531 in New Hanover County south of Wilmington, NC. US-421 is a four-lane divided highway which provides access between Wilmington and the beaches in southern New Hanover County. At the intersection, US-421 has a posted speed limit of 45 mph and a wide turf median. The intersection is TWSC with the stop control on SR-1576/1531. Because the intersection seemed to have a pattern of left-turn leaving collisions related to sight-line obstructions created by opposing left-turn vehicles in the conventional left-turn lanes on US-421, a decision was made to positively offset these left-turn lanes. The project was completed on August 1, 2002 at an estimated cost of $100,000. Figure 99 shows a northbound view of the parallel offset left-turn lanes at this intersection and demonstrates that the lanes are indeed positively offset. Figure 100 shows a southbound view of the parallel offset left-turn lanes at this intersection and demonstrates the difference between the before and after conditions. The top picture in Figure 100 was taken in the after period; however, it was taken from the vantage point of a driver in the former conventional left-turn lane. From this viewpoint, you can get a sense of where the conventional left-turn lanes were located and how a left-turn driver's sight-line might have been obstructed in the presence of opposing left-turn traffic. The bottom picture in Figure 100 shows the view of a driver in the offset left-turn lane and clearly shows an improved sight-line to opposing through traffic.

FIGURE 99. Parallel offset left-turn lanes at US-421 & SR-1576/1531 (northbound view)

FIGURE 100. Parallel offset left-turn lanes – US-421 & SR-1576/1531 (southbound view)

It should be mentioned that the speed limit on US-421 in the vicinity of this intersection was reduced from 55 mph to 45 mph in April of 2002, just prior to the offset left-turn lane installation. This reduced speed zone occurs approximately 500 feet north of the intersection and extends approximately 0.75 miles south. Therefore, the before-after comparison that follows is unfortunately confounded by this speed limit change.

TABLE 34. Offset Left-Turn Lane Before-After Data at US-421 & SR-1576/1531

	BEFORE	AFTER	% CHANGE	SIGNIFICANT DIFFERENCE AT:
ESTIMATED TOTAL ENTERING AADT	26,900	30,800	+ 14.50	
YEARS	3	3		
TOTAL CRASHES	21	12	-42.86	
Crash Frequency/Year	7.00	4.00	-42.86	α = 0.0514 *
Crash Rate/mev	0.71	0.36	-50.09	
FATAL CRASHES	0	0	0	N/A
INJURY CRASHES	16	3	-81.25	
Crash Frequency/Year	5.33	1.00	-81.25	α = 0.0345 *
Crash Rate/mev	0.54	0.09	-83.62	
PDO CRASHES	5	9	+ 80.00	
Crash Frequency/Year	1.67	3.00	+ 80.00	α = 0.1955
Crash Rate/mev	0.17	0.27	+ 57.21	
LEFT-TURN LEAVING CRASHES	12	2	-83.33	
Crash Frequency/Year	4.00	0.67	-83.33	α = 0.0077 *
Crash Rate/mev	0.41	0.06	-85.44	
REAR-END CRASHES (US-421)	0	1	+ Undefined	
Crash Frequency/Year	0	0.33	+ Undefined	α = 0.2113
Crash Rate/mev	0	0.03	+ Undefined	
TOTAL TARGET CRASHES	12	3	-75.00	
Crash Frequency/Year	4.00	1.00	-75.00	α = 0.0107 *
Crash Rate/mev	0.41	0.09	-78.17	
RIGHT-ANGLE CRASHES	7	3	-57.14	
Crash Frequency/Year	2.33	1.00	-57.14	α = 0.1476
Crash Rate/mev	0.24	0.09	-62.57	
OTHER CRASHES	2	6	+ 200.00	
Crash Frequency/Year	0.67	2.00	+ 200.00	α = 0.1667
Crash Rate/mev	0.07	0.18	+ 162.01	

* Significant difference in sample means assuming unequal variances at a 90% level of confidence using a one-tailed t-test.

The before and after crash data at this intersection are presented and compared in Table 34. The three year before period at the intersection includes crash data from 1999 through 2001, while the three year after period includes crash data from 2003 through 2005.

In the before period, there were a total of 21 intersection-related crashes, giving an average crash frequency of 7.0 crashes per year and a crash rate of 0.71 crashes per mev. In the after period, there were a total of 12 intersection-related collisions, giving an average crash frequency of 4.0 crashes per year and a crash rate of 0.36 crashes per mev. Therefore, the annual average crash frequency was reduced by 43 percent and the overall crash rate was reduced by 50 percent. Furthermore, the intersection experienced a considerable decrease in crash severity. Injury crash frequency was reduced by 81 percent, while the injury crash rate was reduced by 84 percent.

Since the offset left-turn lane treatment is meant to reduce left-turn leaving and rear-end crashes on the mainline, these are considered to be the "target" crash types and are examined separately. In the before period, 12 of the 21 crashes (57 percent) were left-turn leaving collisions with no rear-end collisions on US-421. In the after period, 2 of the 12 crashes (17 percent) were left-turn leaving crashes with one rear-end collision on US-421. Therefore, the combination of the offset left-turn treatment and the speed reduction on US-421 reduced the frequency of left-turn leaving collisions by 83 percent and reduced the targeted crash type frequency by 75 percent. Taking into account vehicle exposure, the target crash rate at this intersection was reduced by 78 percent.

Because there were three years of before and after crash data at US-421 and SR-1576/1531, statistical comparison of the before and after mean annual crash frequencies was performed. Using a one-tailed t-test for detecting differences in sample means assuming unequal variances and a 90 percent level of confidence ($\alpha = 0.10$), the mean annual crash frequency was significantly reduced in the after period for total, injury, left-turn leaving, and target collisions as shown in Table 34. The increases in PDO, rear-end, and other collisions were not statistically significant.

The second safety evaluation of an offset left-turn lane installation conducted by the NCDOT was at the intersection of US-421 (Carolina Beach Road) and SR-1524 (Golden Road), which is located approximately 3 miles north of the previous site, yet still south of Wilmington, NC. At this intersection, US-421 is a four-lane divided highway with a posted speed limit of 55 mph; however, this intersection seems to be more suburban in nature than the previous site as the west minor road leg is an entrance to the Masonboro Commons

Shopping Center. In addition, even though this intersection is TWSC, there are signalized intersections located approximately 1/4 mile to the south and 1/3 mile to the north. These traffic signals have the potential to stop opposing US-421 through traffic upstream and create additional gaps for left-turning traffic. Because this intersection seemed to exhibit a pattern of left-turn leaving collisions related to sight-line obstructions created by opposing left-turn vehicles in the conventional left-turn lanes on US-421, a decision was made to positively offset these left-turn lanes. The project was completed on February 4, 2003 at a cost of approximately $95,000. Figure 101 shows a northbound view of the parallel offset left-turn lanes at this intersection and demonstrates the difference between the before and after conditions. Both pictures in Figure 101 were taken in the after period; however, the top picture was taken from the viewpoint of a driver in the former conventional left-turn lane. From this vantage point, you can get a sense of where the left-turn lanes were previously located and how a left-turn driver's sight-line may have been obstructed by opposing left-turn vehicles. The bottom portion of Figure 101 demonstrates the improved sight-line of a driver in the offset left-turn lane.

FIGURE 101. Parallel offset left-turn lanes at US-421 & Golden Road (northbound view)

TABLE 35. Offset Left-Turn Lane Before-After Crash Data at US-421 & Golden Road

	BEFORE	AFTER	% CHANGE	SIGNIFICANT DIFFERENCE AT:
ESTIMATED TOTAL ENTERING AADT	27,400	33,100	+ 20.80	
YEARS	3	3		
TOTAL CRASHES	26	20	-23.08	
Crash Frequency/Year	8.67	6.67	-23.08	α = 0.2857
Crash Rate/mev	0.87	0.55	-36.32	
FATAL CRASHES	1	0	-100	
Crash Frequency/Year	0.33	0	-100	α = 0.2113
Crash Rate/mev	0.03	0	-100	
INJURY CRASHES	19	8	-57.89	
Crash Frequency/Year	6.33	2.67	-57.89	α = 0.1309
Crash Rate/mev	0.63	0.22	-65.15	
PDO CRASHES	6	12	+ 100	
Crash Frequency/Year	2.00	4.00	+ 100	α = 0.0908 *
Crash Rate/mev	0.20	0.33	+ 65.56	
LEFT-TURN LEAVING CRASHES	14	0	-100	
Crash Frequency/Year	4.67	0	-100	α = 0.0424 *
Crash Rate/mev	0.47	0	-100	
REAR-END CRASHES (US-421)	0	4	+ Undefined	
Crash Frequency/Year	0	1.33	+ Undefined	α = 0.0918 *
Crash Rate/mev	0	0.11	+ Undefined	
TOTAL TARGET CRASHES	14	4	-71.43	
Crash Frequency/Year	4.67	1.33	-71.43	α = 0.0642 *
Crash Rate/mev	0.47	0.11	-76.35	
RIGHT-ANGLE CRASHES	8	12	+ 50.00	
Crash Frequency/Year	2.67	4.00	+ 50.00	α = 0.3091
Crash Rate/mev	0.27	0.33	+ 24.17	
OTHER CRASHES	4	4	0	
Crash Frequency/Year	1.33	1.33	0	α = 0.5000
Crash Rate/mev	0.13	0.11	-17.22	

* Significant difference in sample means assuming unequal variances at a 90% level of confidence using a one-tailed t-test.

The before and after crash data for this intersection are summarized and compared in Table 35. The three year before period includes data from 11/1/1999 through 10/31/2002, while the three year after period consists of crash data from 4/1/2003 through 3/31/2006. For the purpose of this analysis, it should be mentioned that the signalized intersection to the north was signalized throughout the before and after periods; however, the signalized intersection to the south was signalized in 2001 (during the before period). Therefore, the crash data during the before period may have been affected by this change as the signal likely changed the arrival distribution of northbound traffic on US-421 through which southbound left-turning traffic must turn.

During the before period, there were a total of 26 intersection-related collisions, giving an average crash frequency of 8.67 crashes annually and a crash rate of 0.87 crashes per mev. In the after period, there were a total of 20 intersection-related collisions, giving an average crash frequency of 6.67 crashes per year and a crash rate of 0.55 crashes per mev. Therefore, there was a 23 percent reduction in annual crash frequency and a 36 percent reduction in the overall crash rate. In addition, the severity of the crashes was lessened as the fatal/injury crash frequency was reduced by 60 percent and the fatal/injury crash rate was reduced by 67 percent.

The intersection also experienced a dramatic reduction in left-turn leaving collisions. In the before period, 14 of the 26 collisions at the intersection (54 percent) were left-turn leaving collisions on US-421. In the after period, no left-turn leaving collisions occurred giving a 100 percent reduction for this targeted crash type. The other targeted crash type (rear-end collisions on US-421) increased from zero in the before period to four in the after period; therefore, the offset left-turn lanes at this intersection reduced the overall target crash type frequency by 71 percent and the target crash type rate by 76 percent.

Because there were three years of before and after crash data at US-421 and Golden Road, statistical comparison of the before and after mean annual crash frequencies was performed using a one-tailed t-test for detecting differences in sample means assuming unequal variances and a 90 percent level of confidence ($\alpha = 0.10$). The results show that the reduction in left-turn leaving collisions was statistically significant which contributed to the significant reduction in the targeted crash types. However, the increases in rear-end and PDO crashes were also statistically significant.

3.9.4 Offset Left-Turn Lanes Summary

The assumed safety benefit of offset left-turn lanes is that they eliminate the sight distance obstruction created by the presence of opposing left-turn vehicles in conventional left-turn lanes, thereby allowing left-turn drivers to make improved gap selection decisions when exiting the expressway. Expressway intersections most likely to benefit from offset left-turn lanes include: 1) intersections with a history of left-turn leaving collisions resulting from opposing left-turn vehicles on the mainline obstructing each other's sight lines, and 2) intersections with large volumes of opposing left-turning traffic leaving the expressway. No volume warrants for their use appear in the AASHTO Green Book (*3*), but the Illinois Bureau of Design and Environment Manual (*91*) requires a left-turn design volume of 60 vph leaving the mainline from each direction combined with a mainline ADT of at least 1500 vpd.

Both of the North Carolina case studies presented here revealed a significant reduction in the frequency of left-turn leaving collisions as a result of offsetting conventional left-turn lanes at TWSC expressway intersections. Table 36 summarizes these results; however, mainline rear-end collisions increased at both sites as Schurr et al. (*25*) had predicted. No definitive conclusions regarding the safety effects of offset left-turn lanes can be drawn from this study due to the limited number of sites and the limitations of the naïve before-after analysis methodology. Further study with more sites and more data is necessary. In addition, the benefits and tradeoffs of parallel versus tapered type designs should be more thoroughly investigated.

TABLE 36. Offset Left-Turn Lane Safety Effectiveness Summary

	US-421 & Seabreeze Road Conventional LTL → Offset % Change	US-421 & Golden Road Conventional LTL → Offset % Change
Overall Crash Frequency/yr	-43 *	-23
Overall Crash Rate/mev	-50	-36
Left-Turn Leaving Crash Frequency/yr	-83 *	-100 *
Left-Turn Leaving Crash Rate/mev	-85	-100
Target Crash Frequency/yr	-75 *	-71 *
Target Crash Rate/mev	-78	-76

** Statistically significant change at 90% confidence level (changes in crash rates were not tested).*

Currently, the Green Book (*3*) does offer limited guidance regarding the design of offset left-turn lanes and many STAs have incorporated offset left-turn lane standards into their design manuals. However, each state seems to have their own unique geometric design configurations and use warrants; therefore, "best practices" should be compiled and incorporated into the Green Book to encourage national design consistency for both parallel and tapered type designs. Furthermore, definitions of the left-turn offset distance (positive and negative offsets) should be added to the Green Book for clarification. McCoy et al. (*90*) and Staplin et al. (*38*) established that the minimum required offset distance is always positive at divided highway intersections. This fact should be stated clearly in the Green Book (*3*) and the MUTCD (*31*) so that STAs stop constructing offset left-turn lanes with negative offsets as shown in Figure 94, which really defeat their intended purpose. Furthermore, the guidance on minimum offset distances provided by Staplin et al. (*38*) in Figure 92 should be added to the Green Book (*3*) and their recommended signing and marking plan for TWSC divided highway intersections with offset left-turn lanes shown in Figure 95 should be added to the MUTCD (*31*). Finally, to ensure offset left-turn lanes are utilized properly, rumble strips could be placed in the gore area with the edge lines painted through the rumble strips to get a vertically painted edge line face. In addition, providing a tapered lane entry with turf surfacing in the median all the way to the median nose could be considered to provide an approaching left-turn driver with a better sense of where the intersection is ultimately located and to provide an opposing through driver with better visual delineation through the intersection.

3.10 FREEWAY-STYLE INTERSECTION GUIDE SIGNING CASE STUDY

3.10.1 Description

To motorists, a rural expressway may appear to be a freeway. As such, expressway drivers may have the same expectations of the expressway as they would have for a freeway/interstate facility. One of these expectations includes full access control (i.e., no at-grade intersections). In addition, many TWSC rural expressway intersections are not easily visible to approaching drivers, particularly from the uncontrolled expressway approaches. Right-angle collisions at TWSC expressway intersections typically occur as a result of poor gap selection by minor road drivers; however, some of these collisions may have been avoidable had the approaching expressway driver been aware of the intersection and been prepared to slow down or take evasive action as necessary. Intersection recognition devices are a category of intersection safety treatments which are meant to improve intersection conspicuity for approaching drivers, allowing them to recognize the intersection as a potential hazard and to proceed through it with caution. Traditionally, when right-angle collisions begin to occur at TWSC rural expressway intersections, these treatments are the first countermeasures to be applied because they are relatively low-cost and easy to deploy.

One intersection recognition strategy is to enhance the signing and delineation along the uncontrolled mainline approaches. These improvements may include advance guide signs, advance street name signs, advance warning signs/flashers, and/or advance pavement markings. FHWA's *Guidelines and Recommendations to Accommodate Older Drivers and Pedestrians* (*38*) encourage such improvements to enhance the driving environment for older drivers. More specifically, their recommendations address letter height and reflectivity on signs as key issues for older drivers. Providing enhanced signing and delineation for intersections is also an intersection safety strategy addressed in NCHRP 500, Volume 5 (*16*) to reduce patterns of right-angle, rear-end, or turning collisions related to a lack of driver awareness of an unsignalized intersection. This strategy should improve intersection safety by alerting mainline drivers to the potential for entering vehicles at an intersection; thereby heightening awareness and improving driver reaction times when conflicts do occur. However, the safety effectiveness of this strategy has not been quantified (*16*).

The specific treatment discussed in this case study consists of deploying enhanced (i.e., freeway-style) guide signs along rural expressways in advance of TWSC intersections with higher volume minor roads or those with a history of right-angle collisions. Intersections with lower volume minor roads could continue to be identified using conventional signage. However, at critical intersections, the use of freeway-style guide signs over conventional ones should enhance an expressway driver's awareness of an intersection as well as their preparedness for potential conflicts should a minor road vehicle select an unsafe gap when entering the intersection. However, care should be taken not to overuse the freeway-style guide signs at TWSC intersections as drivers would likely become accustomed to their presence and fail to respond accordingly; therefore, they should only be used where a specific problem or volume warrant indicates their need (*16*). Currently, no traffic volume, crash experience, or other warrants have been developed indicating when an agency should consider this type of advance signing at rural expressway intersections.

3.10.2 Existing Design Guidance

Section 2E.26 of the MUTCD (*31*) addresses guide signing for at-grade intersections on expressways by stating:

If there are intersections at grade within the limits of an expressway, guide sign types specified in Chapter 2D (Guide Signs – Conventional Roads) should be used. However, such signs should be of a size compatible with the size of other signing on the expressway. Advance guide signs for intersections at grade may take the form of diagrammatic layouts depicting the geometrics of the intersection along with essential directional information.

MUTCD Table 2E-2 (Table 37) gives minimum letter and numeral sizes for expressway guide signs. Of the guide signs described in MUTCD Chapter 2D, many have likely application at rural expressway intersections; however, of these guide signs, only the CROSSOVER (D-13 Series) signs shown in MUTCD Figure 2D-12 (Figure 102) are specifically meant for use at divided highway intersections. Section 2D.51 of the MUTCD (*31*) states:

Crossover signs may be installed on divided highways to identify median openings not otherwise identified by warning or other guide signs. If used, the CROSSOVER sign (D13-1) should be installed immediately beyond the median opening, either on the right side of the roadway or in the median. The Advance Crossover sign (D13-2) may be installed in advance of the CROSSOVER sign to provide advance notice of the crossover. The distance shown on the Advance Crossover sign should be 1 mile, ½ mile, or ¼ mile and it should be installed either on the right side of the roadway or in the median at approximately the distance shown.

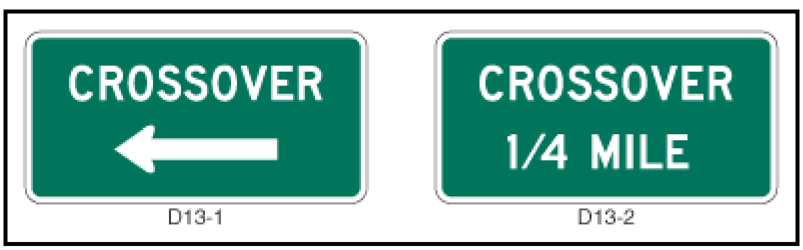


FIGURE 102. CROSSOVER guide signs (from MUTCD Figure 2D-12) (31)

The design and application of diagrammatic signs are described in MUTCD (*31*) Section 2E.19. This section defines a diagrammatic sign as, "Guide signs that show a graphic view of the exit arrangement in relationship to the main highway." However, the guidance in this section only describes their use at interchanges and no discussion or examples of their application for at-grade expressway intersections are provided in the current edition. To find an example of a diagrammatic sign for at-grade intersections, one would have to look in past versions of the manual (see Figure 61B). Examples of diagrammatic signs on expressway intersections can also be found in Nebraska as shown in Figure 103. NDOR has been using

TABLE 37. MUTCD Table 2E-2; Minimum Text Sizes – Expressway Guide Signs (*31*)

Type of Sign	Minimum Size (mm)	Minimum Size (inches)
A. Pull-Through Signs		
Destination — Upper-Case Letters	330	13.3
Destination — Lower-Case Letters	250	10
Route Sign as Message		
Cardinal Direction	250	10
1- or 2-Digit Shield	900 x 900	36 x 36
3-Digit Shield	1125 x 900	45 x 36
B. Supplemental Guide Signs		
Exit Number Word	200	8
Exit Number Numeral and Letter	300	12
Place Name — Upper-Case Letters	265	10.6
Place Name — Lower-Case Letters	200	8
Action Message	200	8
C. Changeable Message Signs		
Characters	265*	10.6*
D. Interchange Sequence Signs		
Word — Upper-Case Letters	265	10.6
Word — Lower-Case Letters	200	8
Numeral	250	10
Fraction	200	8
E. Next X Exits Sign		
Place Name — Upper-Case Letters	265	10.6
Place Name — Lower-Case Letters	200	8
NEXT X EXITS	200	8
F. Distance Signs		
Word — Upper-Case Letters	200	8
Word — Lower-Case Letters	150	6
Numeral	200	8
G. General Services Signs		
Exit Number Word	200	8
Exit Number Numeral and Letter	300	12
Services	200	8
H. Rest Area and Scenic Area Signs		
Word	250	10
Distance Numeral	300	12
Distance Fraction	200	8
Distance Word	250	10
Action Message Word	250	10
I. Reference Location Signs		
Word	100	4
Numeral	250	10
J. Boundary and Orientation Signs		
Word — Upper-Case Letters	200	8
Word — Lower-Case Letters	150	6
K. Next Exit and Next Services Signs		
Word and Numeral	200	8
L. Exit Only Signs		
Word	300	12

*Changeable Message Signs may often require larger sizes than the minimum. A size of 450 mm (18 in) should be used where traffic speeds are greater than 90 km/h (55 mph), in areas of persistent inclement weather, or where complex driving tasks are involved.

FIGURE 103. Application of diagrammatic at-grade intersection signage in Nebraska

this type of advance guide signing at expressway intersections with other US and state highways since 1972; however, they have not conducted any studies regarding their safety effects. One disadvantage of diagrammatic layouts is that it may be difficult to fit street names or multiple shields (if a roadway carries multiple numbered routes) on this type of sign.

Currently, the MUTCD (*31*) guidance for intersection warning signs (Section 2C.37) suggests that, "The relative importance of the intersecting roadways may be shown by different widths of lines in the symbol." However, there is no other guidance given for differentiating the relative importance of one intersection over another using different styles of advance signs. Crash frequencies, crash rates, and traffic volume thresholds can all be used to determine where enhanced signing may be necessary; however, these metrics can all vary widely both within a state and between states. As a result, each state is encouraged to review their own data for guidance relative to implementation warrants. With that being said, an informal review of rural expressway intersections in Minnesota found that most problematic intersections had a minor road volume of at least 2,000 vpd or an expressway volume of at least 25,000 vpd. This indicates that, at either of these volume levels, the demand for gaps is beginning to exceed the number of safe gaps and traffic engineers should consider implementing other safety improvements at the intersection.

3.10.3 Case Studies of Implementation & Safety Effectiveness

3.10.3.1 Minnesota Experience

This case study involves a Mn/DOT signage upgrade project along the US-52 corridor between Rochester and Inver Grove Heights (a suburb of Saint Paul, Minnesota). US-52 is a rural divided expressway functionally classified as a principal arterial (other) with a posted speed limit of primarily 65 mph (the speed limit is lowered to 45 or 55 mph as US-52 passes through or near several small towns). Most intersections along this corridor are at-grade with some interchanges located at higher volume cross roads. The long-range plan is to upgrade the entire corridor into a freeway with full access control; however, the next interchanges scheduled for construction will be at the northern and southern ends of the corridor (near Saint Paul and Rochester) and there are no plans to convert any of the intersections along the middle, more rural portion of the corridor in the near future. In 2006, the ADT along the corridor varied from 17,000 to 43,000 vpd, with the highest volumes near the Saint Paul and Rochester areas.

In 2002, this segment of US-52 was selected for a road safety audit review (RSAR) due to a large number of severe crashes and a RSAR report was completed in February of 2003 (*94*). An observation made by the RSAR Team indicated that drivers on US-52 may have difficulty identifying at-grade intersections. Several factors contributed to this problem including: rolling topography, curvilinear alignment, and vegetation growth. It was also observed that the conventional style guide signing in place at intersections along US-52 (shown in Figure 104 and consistent with MUTCD Chapter 2D guidance) was easy to miss at expressway speeds. The first conventional advance junction route marker (shown in Figure 105A) was typically located ¼ to ½ mile in advance of intersections along US-52 followed by a second conventional route marker with an arrow plaque (shown in Figure 105B) located several hundred feet in advance of the intersections. Intersections with unnumbered routes only included a street name sign posted at the intersection. Destination signage (green guide signs with city names or other destinations) were used at intersections along the corridor as needed.

One countermeasure suggested by the RSAR Team was for Mn/DOT to provide larger advance guide signing at key intersections (similar to guide signs used on freeways) to

FIGURE 104. Conventional advance intersection guide signing along US-52 (94)

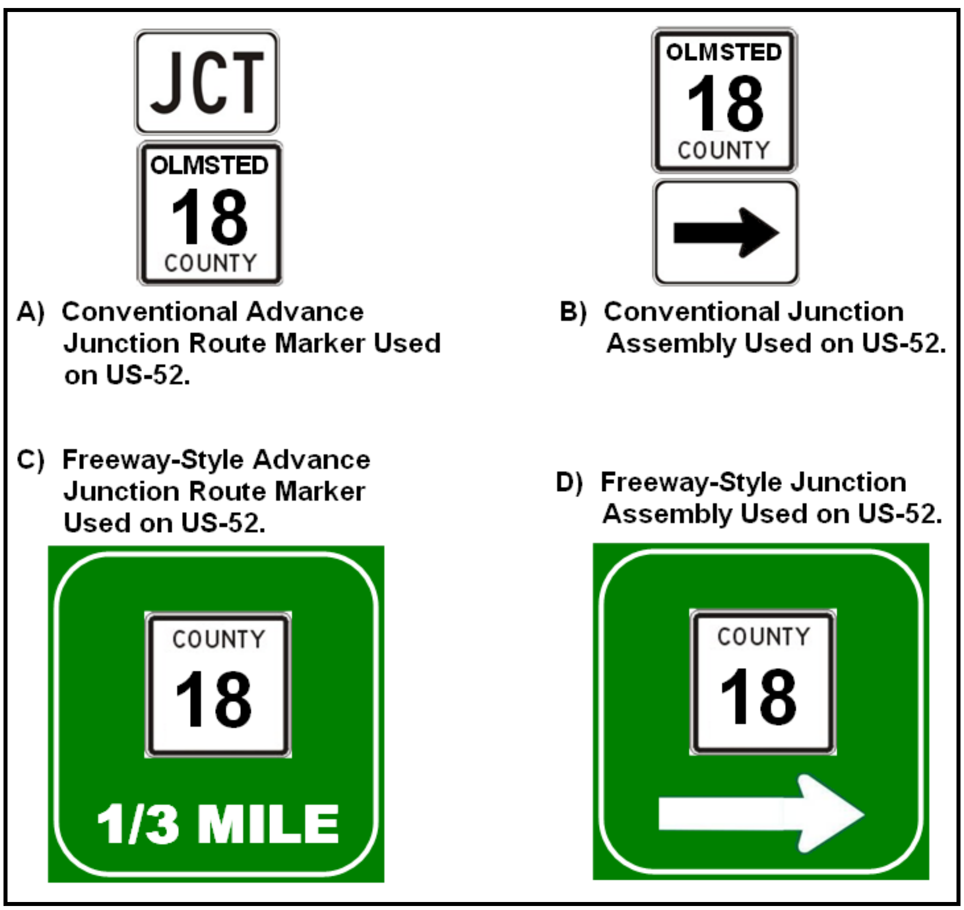


FIGURE 105. Example of intersection guide signing along US-52 (before & after)

inform expressway drivers that they are approaching a major intersection with higher volumes and an increased probability of vehicular conflicts. The RSAR Team believed that this strategy would heighten the awareness of drivers on US-52 at key intersections and would help to prevent right-angle collisions linked to minor road drivers selecting insufficient gaps. As a result of these recommendations, Mn/DOT made guide signing upgrades at eight intersections along US-52 between Rochester and Inver Grove Heights during late 2003/early 2004. This project cost approximately $20,000; however, the work was done by Mn/DOT maintenance forces and this price tag only includes the cost of materials. Six of the eight intersections were located in Goodhue County (CSAH-1 North, CSAH-1 South, CSAH-7, CSAH-9, CSAH-14, and CSAH-68) and the other two were in Olmsted County (CSAH-12 and CSAH-18).

The improvements at these intersections included replacing the conventional advance junction route marker shown in Figure 105A with the freeway-style advance junction route marker shown in Figure 105C and replacing the conventional junction assembly shown in Figure 105B with the freeway-style junction assembly shown in Figure 105D. Example roadway views of these improvements are shown in Figure 106. In some cases, the new signs (very similar to the standard MUTCD CROSSOVER signs shown in Figure 102) were placed further away from the intersections to provide additional advance notice. These new signs provide improved visibility and intersection recognition as compared to the conventional signs since they are larger/brighter and can be seen from much further away. Additional guide sign upgrades have since been implemented at other intersections along US-52 and similar signage improvements for other rural expressway intersections are being developed by Mn/DOT.

Crash data were obtained for all eight intersections along US-52 where the enhanced guide signage was added and a simple before-after comparison was performed for each intersection as well as collectively (see Table 38). The three year before period consists of data from 2000 through 2002, while the two and a half year after period consists of crash data from July 1, 2004 through December 31, 2006. Because there were less than three years of after data available, no statistical comparison was performed. Furthermore, it should be noted that other intersection improvements were recommended by the RSAR Team and Mn/DOT did implement some of those suggestions. For instance, roadway lighting was installed at several intersections along US-52 and the Minnesota State Highway Patrol performed several targeted speed enforcement campaigns throughout the corridor. Therefore, these additional actions make the safety effectiveness of the guide sign enhancements difficult, if not impossible, to determine.

FIGURE 106. Freeway-style advance intersection guide signing along US-52

TABLE 38. Before-After Crash Data Comparison for Freeway-Style Guide Sign Upgrades on US-52

		Goodhue County						Olmsted County		
		CSAH-1 (N. JCT)	CSAH-1 (S. JCT)	CSAH-7	CSAH-9	CSAH-14	CSAH-68	CSAH-12	CSAH-18	TOTALS
BEFORE (3.0 Years)	Total Crashes	5	2	1	20	6	6	24	11	**75**
	Crash Frequency/Yr	1.67	0.67	0.33	6.67	2.00	2.00	8.00	3.67	**25.00**
	Crash Rate/mev	0.24	0.10	0.02	1.02	0.28	0.30	0.83	0.38	**0.42**
	Right-Angle Crashes	2	0	0	14	5	2	13	2	**38**
	Left-Turn Crashes	1	0	0	0	0	0	0	0	**1**
	Right-Turn Crashes	0	0	0	1	0	0	0	0	**1**
	Rear-End Crashes	0	0	0	0	0	3	5	4	**12**
	Ran-Off-Road Crashes	0	0	1	3	1	0	0	0	**5**
	Head-On Crashes	1	1	0	0	0	0	1	0	**3**
	Other Crashes	1	1	0	2	0	1	5	5	**15**
AFTER (2.5 Years)	Total Crashes	5	6	5	11	6	6	20	7	**66**
	Crash Frequency/Yr	2.00	2.40	2.00	4.40	2.40	2.40	8.00	2.80	**26.40**
	Crash Rate/mev	0.30	0.39	0.32	0.73	0.36	0.36	0.81	0.27	**0.45**
	Right-Angle Crashes	1	1	0	5	1	2	10	2	**22**
	Left-Turn Crashes	0	0	0	0	0	0	0	0	**0**
	Right-Turn Crashes	0	0	0	0	0	0	0	0	**0**
	Rear-End Crashes	2	2	0	2	2	3	6	3	**20**
	Ran-Off-Road Crashes	1	0	1	3	3	1	3	1	**13**
	Head-On Crashes	0	0	0	0	0	0	0	1	**1**
	Other Crashes	1	3	4	1	0	0	1	0	**10**
% CHANGE	Total Crash Frequency/Yr	+ 20.00	+260.00	+500.00	-34.00	+ 20.00	+ 20.00	0	-23.64	**+ 5.60**
	Total Crash Rate/mev	+ 25.00	+290.00	+1500.00	-28.43	+ 28.57	+ 20.00	-2.41	-28.95	**+ 7.14**
	Right-Angle Crash Frequency/Yr	-40.00	+Undefined	0	-57.14	-76.00	+ 20.00	-7.69	+ 20.00	**-30.53**
	Rear-End Crash Frequency/Yr	+Undefined	+Undefined	0	+Undefined	+Undefined	+ 20.00	+ 44.00	- 10.00	**+ 100.00**

The annual crash frequency (combined for all eight intersections) was 25.0 crashes per year in the before period, which increased to 26.4 crashes per year in the after period (an increase of 6 percent). The overall crash rate for the eight intersections increased by 7 percent. For the individual intersections, the annual crash frequency and crash rate increased at five of the eight intersections. However, these increases were mainly due to a boost in rear-end and ran-off-road collisions as there was a reduction in right-angle collisions at four of the eight intersections, which is the crash type the countermeasure was meant to address. Right-angle collisions also decreased overall. In the before period, 38 of the 75 collisions (51 percent) were of the right-angle variety. In the after period, the right-angle crash distribution dropped to 33 percent (22 out of 66), which is below the 36 percent right-angle crash distribution average for TWSC rural expressway intersections in Minnesota (*4*). Furthermore, the overall right-angle annual crash frequency decreased from 12.7 to 8.8 collisions per year; a 31 percent decrease after the enhanced guide signage was installed.

3.10.4 Freeway-Style Intersection Guide Signing Summary

The assumed safety benefit of providing enhanced freeway-style guide signs along the expressway in advance of TWSC rural expressway intersections is that they increase intersection conspicuity and increase the distance for intersection recognition; thereby alerting the expressway driver to the presence of the intersection sooner and heightening their awareness should they encounter a minor road vehicle which selects an unsafe gap. Because gap selection becomes more critical as traffic volumes increase, particularly on the minor roads, this strategy should be limited to intersections with higher minor road traffic volumes (although no volume warrants have been developed). By limiting the use of this strategy to critical intersections, the expressway driver can be better prepared for the higher potential of entering minor road traffic and be ready to take evasive action, if necessary at these intersections. This information would also allow expressway drivers to be more prepared should they encounter slower traffic entering or exiting the expressway. As a result, this countermeasure is expected to reduce right-angle and rear-end collisions.

The safety effectiveness of enhanced guide signing determined through the Mn/DOT experience is considered inconclusive due to the fact that there were less than three years of after data at the eight intersections studied and the added roadway lighting and speed enforcement measures along the US-52 corridor confound the analysis. Nonetheless, the case study revealed a 31 percent decrease in the frequency of right-angle collisions, while there was a slight increase in overall crash frequency and rates due in large part to a 100 percent increase in rear-end collisions. Therefore, enhanced freeway-style guide signing may not be able to effectively address the entire crash problem at rural expressway intersections, but it could be one part of an overall intersection safety strategy.

Currently, the MUTCD (*31*) instructs the traffic engineer to use the same types of guide signs specified for use on conventional roads when posting guide signs at rural expressway intersections. Although safety effectiveness and volume warrants have yet to be determined, language could be added to the MUTCD which supports the use of freeway-style advance guide signs (with or without diagrammatic layouts as used in Nebraska) for critical at-grade rural expressway intersections (i.e., those with higher volume minor roads, higher crash rates, or other geometric issues). This approach provides overall sign sizes and letter heights appropriate for the high speeds typically found on rural expressways and certainly enhances the expressway driver's awareness of an upcoming intersection.

3.11 DYNAMIC ADVANCE INTERSECTION WARNING SYSTEMS CASE STUDY

3.11.1 Description

Similar to the enhanced freeway-style guide signs discussed in the previous case study, dynamic advance intersection warning systems are an intersection recognition treatment which is meant to enhance an expressway driver's awareness of an approaching TWSC intersection and the conflicts which may arise. However, dynamic advance intersection warning systems are an ITS application which provides information regarding real-time traffic conditions (i.e., the presence of cross traffic on the minor road or in the median). The systems typically consist of static VEHICLES ENTERING WHEN FLASHING (VEWF) warning signs with traffic actuated flashers on the expressway approaches and in-pavement loop detectors on the minor roads as well as in the median. When traffic is detected on the minor road(s) or in the median, the flashers on the VEWF signs are activated on the expressway approaches; thereby warning expressway drivers that one or more vehicles are present at the intersection and may enter from the minor road. Some of these systems have been set up to concurrently warn drivers on the minor road when there is traffic approaching on the major road, but this addition to the system was not examined in this case study as it has primarily been used at intersections on two-lane highways and is similar to IDS technology described in an earlier case study.

Right-angle collisions at TWSC expressway intersections typically occur as a result of poor gap selection by minor road drivers; however, some of these collisions may be avoidable if the approaching expressway driver were aware of the intersection and prepared to slow down or take evasive action as necessary. Therefore, this strategy aims to reduce right-angle and rear-end collisions by dynamically alerting mainline drivers to the presence of vehicles at the upcoming intersection; thus heightening their awareness and improving their reaction times should the minor road driver select an unsafe gap when entering the intersection. The safety effectiveness of this strategy at TWSC rural expressway intersections has not been examined; however, two prior studies have examined the effectiveness of similar systems at rural two-lane undivided highway intersections in Virginia and Maine (*79, 80*).

During the late 1990's, Hanscom (*79*) conducted a field study to examine the cost effectiveness, the crash reduction potential, and the driver behavior effects of installing a "Collision Countermeasure System" (CCS) at a TWSC intersection on a rural two-lane highway in Virginia. The system consisted of dynamic signs on both the major and minor roads to alert major road drivers to the presence of minor road traffic at the intersection and to warn stopped minor road drivers of approaching major road traffic. Photos of this system are shown in Figure 107.

FIGURE 107. Collision Countermeasure System tested in Virginia (79)

The benefit-cost analysis showed that the CCS would be cost effective if it were able to prevent one right-angle collision per year. In the five year before period, the intersection under investigation averaged 2.6 right-angle collisions per year, while no right-angle collisions occurred during the two year operation of the CCS; therefore, the CCS led to a 100 percent reduction in right-angle collisions at this site and was cost effective. Finally, the study showed significant ($\alpha = 0.05$) speed reductions on the major road approaches when the CCS was active with a significant ($\alpha = 0.001$) reduction in the proportion of vehicles violating the speed limit on the major road. One concern expressed by the Virginia Department of Transportation (VDOT) was that major road drivers would increase their speed in the absence of CCS activation as a result of a perceived sense of security knowing that no traffic was present at the intersection ahead; however, the study data indicated that this did not occur.

In 2001, the Maine Department of Transportation (*80*) evaluated a similar system at a TWSC intersection on a rural two-lane highway; however, only the dynamic signs installed on the minor road approaches were examined in the before-after safety analysis because the dynamic signs on the major road were already in place during the before period. Still, a speed study during the before period showed that the existing dynamic TRAFFIC ENTERING WHEN FLASHING sign did not significantly reduce approaching vehicle speeds on the major road when the flashers were active.

3.11.2 Existing Design Guidance

Chapter 4K of the MUTCD (*31*) addresses the use of flashing beacons and Section 4K.03 discusses their application as warning beacons on intersection approaches stating that, “They may be used on approaches to intersections where additional warning is required or where special conditions exist.” However, the design and application of the VEWF sign and flashers is not specifically addressed. Furthermore, the current edition of the MUTCD provides no guidance indicating when a highway agency should consider this type of advance warning signage at a TWSC intersection of any kind. Therefore, such guidance should be developed and added to the MUTCD. Crash frequencies, crash rates, and traffic volume thresholds can vary widely both within a state and between states. As a result, each state is encouraged to review their own data for guidance relative to implementation warrants.

However, an informal review of TWSC rural expressway intersections in Minnesota found that most problematic intersections had a minor road volume of at least 2,000 vpd or an expressway volume of at least 25,000 vpd. This indicates that, at either of these volume levels, the demand for gaps is beginning to exceed the number of safe gaps and traffic engineers should consider implementing other safety improvements at the intersection.

3.11.3 Case Studies of Implementation & Safety Effectiveness

Two states (North Carolina and Missouri) have used different applications of dynamic advance warning systems at TWSC rural expressway intersections to warn traffic on the expressway that vehicles are present on the minor road. Their experiences with this safety treatment are documented herein.

3.11.3.1 North Carolina Experience

The NCDOT Safety Evaluation Group (*58*) conducted safety evaluations at two high-speed TWSC rural expressway intersections where dynamic advance intersection warning systems were installed. These before-after spot safety evaluations were conducted at the intersections of: 1) US-74/76 and SR-1800 (Blacksmith Road), and 2) US-421 and North Carolina State Highway 210 (NC-210). At both locations, the dynamic advance intersection warning systems consisted of post mounted VEWF signs with actuated flashers placed in the outside and median shoulders on the expressway approaches. Each evaluation is briefly summarized here; however, further details including collision diagrams can be found in the original reports (*95, 96*). The before and after crash data given in the original reports were compared in terms of percent change; however, no statistical analyses were conducted; therefore, additional statistical comparisons were conducted here.

The first safety evaluation conducted by the NCDOT was at the intersection of US-74/76 and SR-1800 (Blacksmith Road) in Columbus County west of Wilmington, NC. An aerial photo of this TWSC intersection is shown in Figure 108. At this location, US-74/76 is a rural four-lane divided highway with a posted speed limit of 55 mph which bypasses Bolton, NC. There are, however 45 mph advisory speed plaques placed underneath intersection ahead warning signs (W2-1) on both US-74/76 approaches in advance of this intersection. Blacksmith Road is a two-lane undivided roadway which includes channelized right-turn lanes and dually posted stop signs on both intersection approaches. The northbound approach has a posted speed limit of 35 mph, while the southbound approach has a posted speed limit of 55 mph. The median is relatively narrow (as shown in Figure 108) and is yield controlled. The dynamic advance intersection warning signs were placed on the US-74/76 approaches on June 27, 1997 after fatal and other severe injury crashes had occurred at the intersection. A roadway view of the treatment is pictured in Figure 109.

FIGURE 108. Aerial photo of US-74/76 & SR-1800 (Blacksmith Road)

FIGURE 109. Dynamic VEWF signs on eastbound US-74/76 at SR-1800 (Blacksmith Rd)

Before and after crash data are summarized and compared in Table 39. The seven year before period includes crash data from 1990 through 1996, while the seven year after period includes data from 1998 through 2004. It should be noted that the double yellow median centerline and median yield markings shown in Figure 108 were initially painted in 2001 (during the after period) to encourage minor road traffic to use a two-stage gap selection process and to provide them with a better indication of where to stop in the median while waiting for a gap in the far-side intersection. This was done in an attempt to reduce the pattern of far-side right-angle collisions which were occurring. Unfortunately, this additional treatment confounds the analysis, making the safety effectiveness of the dynamic advance warning signs on US-74/76 more difficult to determine.

TABLE 39. VEWF Signs Before-After Crash Data (US-74/76 & SR-1800)

	BEFORE	AFTER	% CHANGE	SIGNIFICANT DIFFERENCE AT:
ESTIMATED TOTAL ENTERING AADT (vpd)	7,700	10,100	+ 31.17	
YEARS	7	7		
TOTAL CRASHES	35	14	-60.00	
Crash Frequency/Year	5.00	2.00	-60.00	$\alpha = 0.0023$ *
Crash Rate/mev	1.78	0.54	-69.50	
FATAL CRASHES	2	1	-50.00	
Crash Frequency/Year	0.29	0.14	-50.00	$\alpha = 0.2764$
Crash Rate/mev	0.10	0.04	-61.88	
INJURY CRASHES	20	8	-60.00	
Crash Frequency/Year	2.86	1.14	-60.00	$\alpha = 0.0332$ *
Crash Rate/mev	1.02	0.31	-69.50	
PDO CRASHES	13	5	-61.54	
Crash Frequency/Year	1.86	0.71	-61.54	$\alpha = 0.0207$ *
Crash Rate/mev	0.66	0.19	-70.68	
RIGHT-ANGLE CRASHES	30	13	-56.67	
Crash Frequency/Year	4.29	1.86	-56.67	$\alpha = 0.0106$ *
Crash Rate/mev	1.52	0.50	-66.96	
Far-Side Right-Angle	29	12	-58.62	
Crash Frequency/Year	4.14	1.71	-58.62	$\alpha = 0.0109$ *
Crash Rate/mev	1.47	0.47	-68.45	
Near-Side Right-Angle	1	1	0	
Crash Frequency/Year	0.14	0.14	0	$\alpha = 0.5000$
Crash Rate/mev	0.05	0.04	-23.76	
OTHER CRASHES	5	1	-80.00	
Crash Frequency/Year	0.71	0.14	-80.00	$\alpha = 0.0536$ *
Crash Rate/mev	0.25	0.04	-84.75	

* Significant difference in sample means assuming unequal variances at a 90% level of confidence.

In the before period, there were a total of 35 intersection-related crashes (2 fatal, 20 injury, and 13 PDO), giving a crash frequency of 5.0 crashes per year and a crash rate of 1.78 crashes per mev. Of the 35 total crashes, 30 (86 percent) were of the right-angle variety and 29 were far-side right-angle collisions. In the after period, there were a total of 14 intersection-related collisions (1 fatal, 8 injury, and 5 PDO), giving a crash frequency of 2.0 crashes per year and a crash rate of 0.54 crashes per mev. Therefore, the annual crash frequency was reduced by 60 percent and the crash rate declined 70 percent after the dynamic advance intersection warning signs/flashers were installed on US-74/76. Furthermore, the annual right-angle crash frequency declined by 57 percent and the right-angle crash rate was reduced by 67 percent. However, the distribution of right-angle and fatal/injury crashes remained nearly the same in the after period (around 90 and 65 percent, respectively).

Because there were more than three years of before and after data at this location, statistical comparison of the before and after mean annual crash frequencies was performed using a one-tailed t-test for detecting differences in sample means assuming unequal variances and a 90 percent level of confidence ($\alpha = 0.10$). The results show that the reductions in total, injury, PDO, right-angle, far-side right-angle, and other crashes were statistically significant.

A similar statistical analysis was also performed to compare the 1998 through 2000 after crash data with the 2002 through 2004 after crash data to see if the median pavement markings placed in 2001 had any effect on the intersection's crash experience. Pre-markings, the intersection experienced 2.67 crashes per year with 2.33 being right-angle collisions. Post markings, the intersection experienced 2.0 crashes per year with all of them being right-angle collisions. Therefore, the annual crash frequency was reduced by 25 percent while the right-angle annual crash frequency was reduced by 14 percent; however, these reductions were not statistically significant ($\alpha = 0.10$) so it can be concluded that the overall crash reductions in the seven year after period had more to do with the installation of the dynamic advance warning system.

The second safety evaluation of a dynamic advance intersection warning system conducted by the NCDOT was at the intersection of US-421 and NC-210, which is located north of Wilmington, NC in Pender County. An aerial photo of this TWSC intersection is shown in Figure 110. At this location, US-421 is a rural four-lane divided highway and NC-210 is a two-lane highway; however, both roadways have posted speed limits of 55 mph. In addition, both of the NC-210 approaches have splitter islands and are controlled by dually posted stop signs, while the median is uncontrolled. As a result of a persistent pattern of severe far-side right-angle collisions, dynamic advance VEWF warning signs were installed on the US-421 approaches on February 17, 1998. The system was similar to the one installed at the US-74/76 and SR-1800 intersection shown in Figure 109.

FIGURE 110. Aerial photo of US-421 & NC-210

The before and after crash data for the dynamic advance intersection warning system installed at US-421 and NC-210 are summarized and compared in Table 40. In the five year before period (1993 through 1997), there were a total of 19 intersection-related collisions (2 fatal, 9 injury, and 8 PDO), giving a crash frequency of 3.80 crashes per year and a crash rate of 1.89 crashes per mev. Of the 19 total crashes, 13 (68 percent) were right-angle collisions, 12 of which were far-side collisions. In the five year after period (11/1/1998 through 10/31/2003), there were a total of 11 intersection-related collisions (10 injury and 1 PDO), giving a crash frequency of 2.20 crashes per year and a crash rate of 0.87 crashes per mev. Therefore, the annual crash frequency was reduced by 42 percent and the crash rate was reduced by 54 percent after the VEWF signs were installed on US-421. Furthermore, even though the distribution of right-angle crashes increased to 82 percent, the annual right-angle crash frequency was reduced by 31 percent and the right-angle crash rate was reduced by 45 percent.

TABLE 40. VEWF Signs Before-After Crash Data (US-421 & NC-210)

	BEFORE	AFTER	% CHANGE	SIGNIFICANT DIFFERENCE AT:
ESTIMATED TOTAL ENTERING AADT (vpd)	5,500	6,900	+ 25.45	
YEARS	5	5		
TOTAL INTERSECTION CRASHES	19	11	-42.11	
Crash Frequency/Year	3.80	2.20	-42.11	α = 0.0838 *
Crash Rate/mev	1.89	0.87	-53.85	
FATAL CRASHES	2	0	-100	
Crash Frequency/Year	0.40	0	-100	α = 0.0889 *
Crash Rate/mev	0.20	0	-100	
INJURY CRASHES	9	10	+ 11.11	
Crash Frequency/Year	1.80	2.00	+ 11.11	α = 0.3982
Crash Rate/mev	0.90	0.79	-11.43	
PDO CRASHES	8	1	-87.50	
Crash Frequency/Year	1.60	0.20	-87.50	α = 0.0389 *
Crash Rate/mev	0.80	0.08	-90.04	
RIGHT-ANGLE CRASHES	13	9	-30.77	
Crash Frequency/Year	2.60	1.80	-30.77	α = 0.1745
Crash Rate/mev	1.30	0.71	-44.82	
Far-Side Right-Angle	12	5	-58.33	
Crash Frequency/Year	2.40	1.00	-58.33	α = 0.0040 *
Crash Rate/mev	1.20	0.40	-66.79	
Near-Side Right-Angle	1	4	+ 300.00	
Crash Frequency/Year	0.20	0.80	+ 300.00	α = 0.1875
Crash Rate/mev	0.10	0.32	+ 218.84	
OTHER CRASHES	6	2	-66.67	
Crash Frequency/Year	1.20	0.40	-66.67	α = 0.1308
Crash Rate/mev	0.60	0.16	-73.43	

* Significant difference in sample means assuming unequal variances at a 90% level of confidence using a one-tailed t-test.

Because there were more than three years of before and after crash data at this location, statistical comparison of the before and after mean annual crash frequencies was performed using a one-tailed t-test for detecting differences in sample means assuming unequal variances and a 90 percent level of confidence ($\alpha = 0.10$). The results show that the reductions in total, fatal, PDO, and far-side right-angle collisions were statistically significant. However, the reduction in right-angle collisions overall was not significant due to the increase in near-side right-angle collisions.

3.11.3.2 Missouri Experience

MoDOT has also used dynamic advance intersection warning systems as a collision countermeasure at TWSC rural expressway intersections; however, the system in Missouri is slightly different than the one used in North Carolina. The system installed by MoDOT was deployed at two consecutive intersections along US-54 as it bypasses Linn Creek, MO in Camden County. US-54 in this area is a rural four-lane divided expressway with a posted speed limit of 65 mph. The two intersecting roadways are State Route V and County Road 54-68 which are both two lane undivided roadways that provide direct access to the small town of Linn Creek. Both of these roadways are stop-controlled at their intersections with US-54 while the medians are yield controlled.

At these intersections, the dynamic advance warning system consists of loop detectors placed on the Route V and County Road 54-68 approaches, as well as within the medians, to detect the presence of vehicles waiting to enter or cross US-54. When such a vehicle is detected, flashing beacons on post-mounted WATCH FOR ENTERING TRAFFIC (WFET) and intersection ahead warning signs (W2-1) are activated to inform drivers on US-54 that they should use caution as they approach the intersections because minor road vehicles are present. The WFET signs/flashers are placed on the outside shoulders of US-54 prior to encountering the first of the two intersections in either direction of travel, while the intersection ahead signs/flashers are placed on the outside shoulders and in the median between the two intersections for both directions of travel as shown in Figure 111. MoDOT installed this system at this location in August of 2004.

In the five year before period (1999 through 2003), the intersection of US-54 and Route V experienced a total of 29 collisions (1 fatal, 9 injury, and 19 PDO), giving an average crash frequency of 5.80 crashes per year and a crash rate of 0.65 crashes per mev. The ADT volumes on US-54 and Route V were approximately 23,000 and 1,300 vpd, respectively; therefore, the intersection's crash experience was just above the expected annual crash frequency of 5.58 crashes per year predicted by the Maze et al. (*2*) SPF given in Table 3. Over this same five year time frame, the intersection of US-54 and County Road 54-68 experienced a total of 22 collisions (1 fatal, 9 injury, and 12 PDO), giving an average crash frequency of 4.40 crashes per year.

Crash data during the after period was only available through March of 2005; therefore, the safety effectiveness of the MoDOT dynamic advance intersection warning system was not examined. However, in the seven months following installation, only one collision occurred at each intersection. At Route V, the crash was a PDO collision which occurred in the median. At County Road 54-68, there was a right-angle injury collision in which a southbound vehicle on US-54 struck the back end of a semi-trailer.

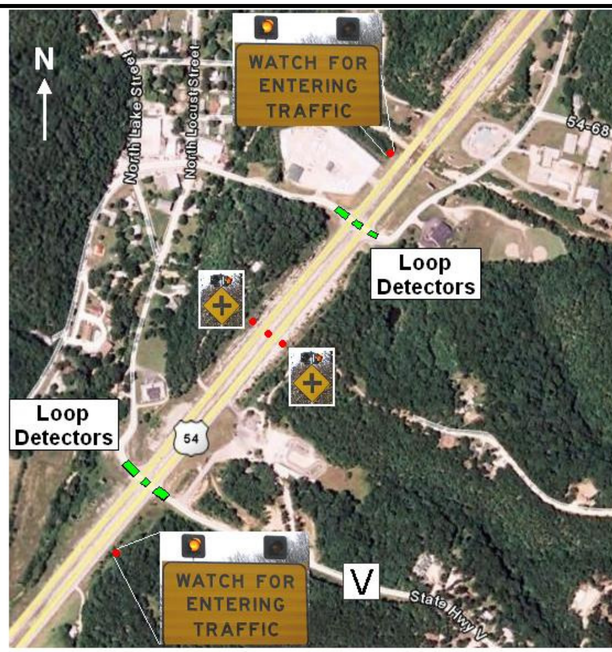


FIGURE 111. Dynamic advance intersection warning system deployed by MoDOT

3.11.4 Dynamic Advance Intersection Warning Systems Summary

The assumed safety benefit of providing dynamic advance intersection warning signs/flashers along the expressway approaches at TWSC rural expressway intersections is that they increase intersection conspicuity and alert the expressway driver to the actual presence of minor road traffic; thereby heightening the expressway driver's awareness and improving their reaction time should a minor road driver select an unsafe gap when entering the intersection. As a result, this strategy targets right-angle and rear-end collisions on the mainline. Such a system may also reduce the speed of expressway vehicles in the presence of minor road traffic (*79*) and reduce crash severity.

The safety effectiveness of this strategy was examined at two locations in North Carolina and the results are summarized in Table 41. Both locations were skewed TWSC intersections with a large proportion of far-side right-angle collisions, which is the typical crash problem at conventional rural expressway intersections. Both sites experienced statistically significant reductions in overall annual crash frequency and, although the distribution of right-angle collisions remained high after the dynamic advance intersection warning systems were installed, the right-angle crash frequency was reduced at both locations (significantly reduced at one site). Furthermore, crash severity was reduced at both locations, demonstrating that this strategy can be an effective safety countermeasure at TWSC rural expressway intersections. However, given the limited number of sites and the shortcomings of the naïve before-after crash analysis methodology, definitive conclusions regarding the safety effectiveness of this strategy cannot be exclusively drawn from this study.

TABLE 41. Dynamic Intersection Warning Systems Safety Effectiveness Summary

	US-74/76 & SR-1800 % Change	US-421 & NC-210 % Change
Overall Crash Frequency/Year	-60 *	-42 *
Overall Crash Rate/mev	-70	-54
Right-Angle Crash Frequency/Year	-57 *	-31
Right-Angle Crash Rate/mev	-67	-45
Fatal Crash Frequency/Year	-50	-100 *
Fatal Crash Rate/mev	-62	-100
Injury Crash Frequency/Year	-60 *	+ 11
Injury Crash Rate/mev	-70	-11

** Statistically significant change at 90% confidence level (changes in crash rates were not tested).*

Currently, the MUTCD (*31*) states that warning beacons may be used on intersection approaches where additional warning is required or where special conditions exist; however, the use and application of dynamic advance intersection warning systems as defined in this case study are not described. Therefore, based on the positive results of the North Carolina experience, it is recommended that design guidance on this intersection safety strategy be included in the MUTCD (*31*) and volume or crash experience warrants be developed to guide highway agencies as to when such a countermeasure should be considered. There is likely a lower volume threshold at which safety begins to deteriorate and the system should be installed, as well as an upper volume limit where the minor road detection loops are not necessary and the mainline flashers should be set to flash continuously.

CHAPTER 4: CONCLUSIONS, RECOMMENDATIONS, & FUTURE RESEARCH NEEDS

4.1 CONCLUSIONS

A rural expressway is a high-speed, multi-lane, divided highway with partial access control which may consist of both at-grade intersections and grade separated interchanges. Many states are converting rural two-lane undivided highways into expressways for improved safety and mobility; however, at-grade intersection collisions on rural expressways are reducing the safety benefits that should be achieved through conversion. Right-angle collisions (particularly on the far-side) are the predominant crash type at conventional TWSC rural expressway intersections. The underlying cause of these collisions in most cases is the inability of the driver stopped on the minor road approach or in the median to judge the arrival time of approaching expressway traffic; therefore, assisting minor road drivers with gap selection is critical to improving safety at TWSC rural expressway intersections.

Traditionally, when the safety performance of these intersections begins to deteriorate, STAs follow the reactive countermeasure application path illustrated in Figure 10 which starts with several low cost signing, marking, or lighting improvements, followed by signalization, and ultimately grade separation. However, highway designers must have other options because the high cost of interchanges limits their use on expressways and TWSC rural expressway intersections often experience safety problems long before traffic signal volume warrants are met. In addition, signals hamper the mobility expressways are meant to provide and they don’t always improve safety as intended. Therefore, the objective of this thesis was to review and document alternative expressway intersection safety countermeasures implemented by various STAs and evaluate their safety effectiveness, where possible, using a naïve before-after analysis methodology in an attempt to begin to understand their potential as rural expressway intersection safety treatments.

The case studies presented in Chapter 3 of this thesis reveal that there are promising safety treatment options for TWSC rural expressway intersections (J-turn intersections, offset T-intersections, MALs, offset turn lanes, enhanced guide signing, and dynamic intersection warning systems) which help to address the gap selection issue while avoiding signalization and grade separation. The case studies help us begin to understand the safety improvement potential of these countermeasures and set the stage for the development of a richer set of design options as shown in Figure 112. However, because sufficient sample sizes were not available for any of the case studies, future research is necessary to more specifically determine the safety effectiveness of these non-traditional designs as well as the conditions under which each countermeasure should be considered and under which each one would be expected to fail in terms of safety and/or operations.

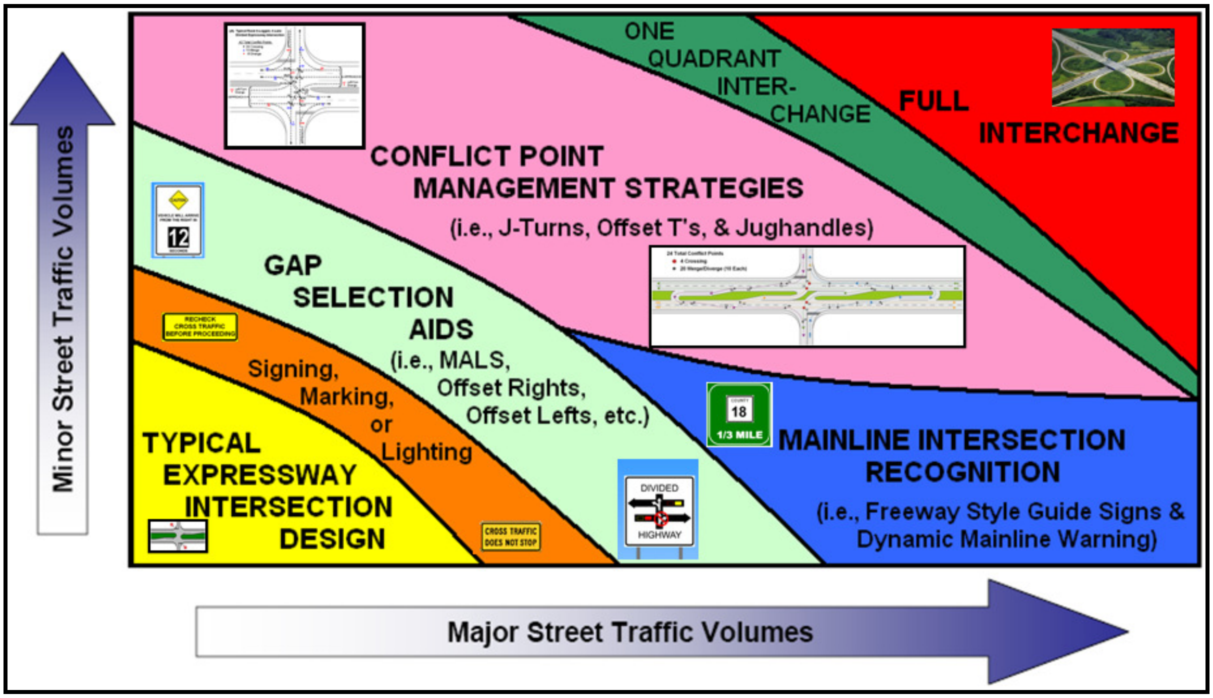


FIGURE 112. Updated TWSC rural expressway intersection countermeasure matrix

In addition, reviews of the Green Book (*3*) and the MUTCD (*31*) revealed many possible areas where these national guides are lacking in terms of design guidance for these non-traditional expressway intersection designs. Modifications to the Green Book and the MUTCD have been suggested throughout this thesis. However, modifications to the Green Book must be made through the AASHTO Technical Committee on Geometric Design and changes to the MUTCD must be made through FHWA’s MUTCD Team and the National Committee on Uniform Traffic Control Devices. The recommendations provided are meant for the consideration of these groups and it is ultimately their responsibility to actually modify the contents of those manuals. Furthermore, the safety effectiveness of the countermeasures discussed in the case studies can only be determined if STAs are willing to deploy and rigorously evaluate them. Therefore, the recommendations that follow are specific, but made to others who must implement them for this research to have any impact on rural expressway intersection design and safety.

4.2 RECOMMENDATIONS

4.2.1 Recommendations for Design Guidance & The AASHTO Green Book (*3*)

As a portion of the literature review for each case study in this thesis, a review of the design guidance within the 2004 AASHTO Green Book (*3*) was conducted for each treatment. As a result, areas where the existing guidance is lacking or is inconsistent with the latest safety research and STA practice were identified and the recommendations for change have been separated into three general categories: organizational changes, philosophical changes, and design guidance updates. The recommendations within these three areas are briefly summarized here.

4.2.1.1 Organizational Changes

Design guidance for rural expressways is poorly organized and spread throughout several chapters of the current edition of the Green Book which may create confusion for roadway designers. This is most likely due to the fact that expressways are a hybrid design with some elements designed like freeways, while other elements (particularly intersections) are designed similar to rural arterial highways. The most ideal solution would be to reorganize the Green Book so that all material on rural expressways and rural expressway intersections is included in a single comprehensive chapter. However, reorganizing the Green Book for expressways would likely be a tremendous undertaking while the modifications might not address all of the issues and may create other confusion in using what is already a cumbersome guide and reference manual. Although it is believed that, at a minimum, Chapter 9 of the Green Book should be revised to include a separate section on expressway intersection design, a more realistic approach may be to create a separate manual for expressway design similar to ITE's "*Freeway and Interchange Geometric Design Handbook*" (*97*). Because expressways do not have a rich history of guidance and literature like freeways, the first addition may address many of the design issues identified in this thesis and map out future research needs. Once the expressway guidance document becomes more mature, the most essential information it contains could be incorporated into the Green Book.

4.2.1.2 Philosophical Changes

Rural expressway corridors typically outlast their at-grade intersections in terms of both safety and operational efficiency as highway designers are usually unable to effectively perpetuate the initial design of an expressway intersection beyond ten years. When the safety performance of an at-grade expressway intersection begins to deteriorate, countermeasures are considered at that time. This current philosophy is therefore reactive and problematic as the appropriate countermeasures may take years to develop while the safety issues continue to occur. Therefore, the Green Book could incorporate a more proactive expressway intersection safety planning process with triggers defining when to start planning for or constructing the next level of intersection design as a conventional TWSC intersection transitions into a full interchange over the course of its life cycle (see Figure 112). In addition, expressway intersection safety needs to be more actively considered during the initial expressway corridor planning process through the strategic placement of intersections on tangent sections, the reduction of intersection skew, the use of frontage roads, and the use of offset T-intersections along the corridor.

4.2.1.3 Design Guidance Updates

The recommendation that seems the most important is to update the Green Book to include the rural expressway intersection designs which address the issue of gap selection for minor road drivers. Currently, little or no design guidance is available in the Green Book regarding J-turn intersections, offset T-intersections, MALs, or offset right-turn lanes. Consequently, few STAs are using these designs. Furthermore, the existing design guidance for jughandle intersections, median U-turn intersections, and offset left-turn lanes is extremely limited and needs to be updated to reflect current safety research and the lessons learned from the case studies presented in this thesis. Once more research has been conducted, the Green Book could eventually address the conditions under which each design is warranted to more fully develop the countermeasure matrix shown in Figure 112.

4.2.2 Recommendations for Design Guidance & The MUTCD (*31*)

As a portion of the literature review for each case study within this thesis, a review of the signing and marking guidance within the 2003 Edition of the MUTCD (*31*) was conducted for each treatment. As a result, areas where the existing guidance is insufficient or inconsistent with the latest safety research and STA practice were identified and the recommendations for improvement were separated into three general categories: assistance for minor road drivers, assistance for expressway drivers, and other technical corrections. The recommendations within these three areas are briefly summarized here.

4.2.2.1 Assistance for Minor Road Drivers

A primary enhancement to the current MUTCD (*31*) guidance for TWSC rural expressway intersections would be to identify and incorporate any traffic control devices or markings which would assist minor road drivers with their decision-making processes for judging and selecting safe gaps in the expressway traffic stream. Currently, the MUTCD does not address the need for or the application of such devices and/or markings. Even though there is no widely accepted device to assist with gap selection from the minor road, there have been attempts to develop and deploy experimental systems such as IDS, static roadside markers, median pavement markings, and median signage. These devices are meant to inform minor road drivers of the size and availability of gaps in expressway traffic, encourage a two-stage gap selection process, and/or remind them to look both ways. The MUTCD should provide some guidance and national uniformity for the use of such devices.

4.2.2.2 Assistance for Expressway Drivers

Another enhancement to the current MUTCD (*31*) would be to include language which supports the use of intersection recognition strategies on the expressway approaches (i.e., freeway-style guide signs, diagrammatic guide signs, dynamic warning signs and flashers, or other such devices) to help expressway drivers identify TWSC intersections with a higher crash risk so that they might apply extra caution. As pointed out throughout this thesis, not all TWSC expressway intersections have the same crash risk. The relative safety of an intersection depends on many factors, but skewed intersections, intersections where the mainline is on a horizontal or vertical curve, intersections with high minor road volumes, intersections with extreme hourly peaking on the minor road, or intersections with some combination of the above tend to have higher crash frequencies. These characteristics seem to make it more difficult for minor road drivers to select safe gaps. Although these strategies would not aid minor road drivers with gap selection directly in this regard, it would alert the expressway driver to the increased potential for conflict so that they might be more prepared to take evasive action if necessary should a minor road driver select an unsafe gap. The existing MUTCD guidance calls for similar guide signs to be used on expressways as used on conventional roads and no examples of diagrammatic signage for at-grade intersections are provided. In addition, the use of dynamic mainline warning devices such as VEWF or WFET signs with flashers is not described. Therefore, future MUTCD additions could include mainline intersection recognition signage which alerts expressway drivers to the presence of intersections with an elevated crash risk.

4.2.2.3 Other Technical Corrections

A number of technical corrections to the current MUTCD (*31*) guidance were identified. The recommended corrections are possible without any further research or development. The technical corrections can be grouped into three areas: figure modifications, figure additions, and correcting inconsistencies which exist between the MUTCD and the Green Book (*3*). The major figure modification involves changing MUTCD Figure 2B-15 (Figure 93) so that it illustrates positively offset left-turn lanes. The recommended figure additions include adding figures for: 1) standard signing and marking at a conventional TWSC expressway intersection with a median width of 30 feet or more and offset left-turn lanes, 2) warning and/or guide signing standards for conventional TWSC expressway intersections with elevated crash risk, and 3) standard signing and marking for the non-traditional expressway intersection designs (i.e., J-turn intersections, offset T-intersections, jughandle intersections, MALs, and offset right-turn lanes). Signing standards for the non-traditional expressway intersection designs are necessary for the sake of nationwide uniformity and may require coordination with STAs who have already experimented with these designs and developed their own standard signing plans. Finally, inconsistencies in the median width definition and the minimum median storage requirements need to be cleared up between the MUTCD and the Green Book.

4.3 FUTURE RESEARCH NEEDS

Chapter 3 of this thesis contains ten case studies of innovative TWSC rural expressway intersection treatments which are believed to have potential in improving intersection safety. Naïve before-after crash data comparisons were performed for seven of the ten treatments examined (no data were available for the other three) and most illustrated improved overall safety and/or a reduction in the targeted crash types. However, a limited number of sites were examined and, in some cases, the amount of before and after crash data was inadequate to perform any statistical evaluation. Furthermore, the naïve before-after analysis methodology does not take regression-to-the-mean into account and it is not known exactly what part of the noted change in safety can be attributed to the treatment and what part may be due to changes in other external factors. Therefore, according to the NCHRP 500, Volume 5 (*16*) definition, these intersection treatments are still considered to be either "tried" or "experimental" and need to be properly evaluated in order to move them into the "proven" category.

In some cases, it was hard to believe that more implementation of the innovative designs has not occurred and that evaluations of their effects could not be found. For example, offset left-turn lane design guidance is included in the Green Book, the MUTCD, and a number of STA design manuals. Therefore, with guidance this widely available, it was anticipated that a number of examples of implementation and evaluations of their safety effects would be found. Unfortunately, it seems relatively few STAs have used offset left-turn lanes at TWSC rural expressway intersections and, of those that have, only one was able to provide data examining their safety effects. It is believed that experimentation with the innovative safety strategies is being hampered by the lack of substantial proof that they actually do improve safety without creating other operational issues. The only way to break out of this vicious cycle of in-action is for a group of STAs (or all STAs) to agree upon innovative intersection design standards, develop a data collection protocol, build several at appropriate locations for a statistically sufficient sample, monitor operations and collect data for a sufficient time period, and perform valid statistical evaluations. Therefore, it is recommended that a pooled fund study be organized and initiated through a national body (such as FHWA or AASHTO), and managed through the Transportation Research Board (TRB) which would start to deploy and rigorously evaluate some or all of the innovative rural expressway intersection treatments discussed in this thesis. With solid evidence of the safety improvement of these treatments, more STAs would be willing to support their implementation. In addition to determining their safety effects, more research is also necessary to determine the conditions under which each treatment should be considered and under which each would be expected to fail in terms of safety or operations.

REFERENCES

1) McDonald, J.W., "Relation Between Number of Accidents and Traffic Volume at Divided-Highway Intersections." *Highway Research Board Bulletin*, No. 74 (1953) p. 7-17.

2) Maze, T.H., N.R. Hawkins, and G. Burchett, *Rural Expressway Intersection Synthesis of Practice and Crash Analysis: Final Report*, Center for Transportation Research and Education, Iowa State University, Ames, Iowa (October 2004) 129 pp. http://www.ctre.iastate.edu/reports/expressway.pdf. (As of May 17, 2007).

3) American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets, Fifth Edition*, AASHTO, Washington, D.C. (2004) 940 pp.

4) Preston, H., R. Storm, M. Donath, and C. Shankwitz, *Review of Minnesota's Rural Intersection Crashes: Methodology for Identifying Intersections for Intersection Decision Support (IDS), Report #1 in the Series: Developing Intersection Decision Support Solutions*, CH2M Hill, University of Minnesota, Minnesota Department of Transportation, St. Paul, MN (May 2004) 44 pp. http://www.lrrb.org/pdf/200431.pdf. (As of August 16, 2007).

5) Federal Highway Administration Office of Highway Policy Information, *Highway Statistics Annual Publications*, http://www.fhwa.dot.gov/policy/ohpi/hss/hsspubs.htm. (As of May 17, 2007).

6) Missouri Department of Transportation Engineering Policy Group, "Rural Expressway Median Openings." *MoDOT Technical Bulletin*, No. 1 (May 2006). http://www.modot.org/business/documents/TechnicalBulletinNo.1May06.pdf. (As of May 17, 2007).

7) Nebraska Department of Roads, *NDOR Roadway Design Manual*, Lincoln, NE (July 2006) 674 pp. http://www.nebraskatransportation.org/roadway-design/manual.htm. (As of May 21, 2007).

8) Nebraska Department of Roads Highway Safety Division, *Accident Experience of Rural Highways: Two-Lane versus Four-Lane Expressways*, unpublished, Lincoln, NE (February 2000).

9) Harwood, D.W., M.T. Pietrucha, M.D. Wooldridge, R.E. Brydia, and K. Fitzpatrick, *NCHRP Report 375: Median Intersection Design*, TRB, National Research Council, Washington, D.C. (1995) 99 pp.

10) Iowa Department of Transportation Office of Systems Planning, *Iowa In Motion: State Transportation Plan*, Iowa Department of Transportation, Ames, Iowa (July 15, 1997) 89 pp. http://www.dot.state.ia.us/transplan.pdf. (As of May 24, 2007).

11) Priest, R.V., "Statistical Relationships Between Traffic Volume, Median Width, and Accident Frequency On Divided-Highway Grade Intersections." *Highway Research News*, No. 13 (June 1964) pp. 9-20.

12) Bonneson, J.A., and P.T. McCoy, *Interchange vs. At-Grade Intersection on Rural Expressways, Final Report*, Nebraska Department of Roads Project No. TRP-02-25-91, University of Nebraska-Lincoln, Lincoln, Nebraska (May 1992) 50 pp.

13) Maze, T.H., H. Preston, R.J. Storm, N. Hawkins, and G. Burchett, "Safety Performance of Divided Expressways." *ITE Journal*, Vol. 75, No. 5 (May 2005) pp. 48-53.

14) Burchett, G.D., and T.H. Maze, "Rural Expressway Intersection Characteristics as Factors in Reducing Safety Performance." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1953 (2006) pp. 71-80.

15) Eck, R.W., and G. Winn, "Older Driver Perception of Problems at Unsignalized Intersections on Divided Highways." *Transportation Research Record: Journal of the Transportation Research Board*, No. 1818 (2002) pp. 70-77.

16) Neuman, T.R., R. Pfefer, K.L. Slack, K.K. Hardy, D.W. Harwood, I.B. Potts, D.J. Torbic, and E.R.K. Rabbani, *NCHRP Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan, Volume 5: A Guide for Addressing Unsignalized Intersection Collisions*, Transportation Research Board, Washington, D.C. (2003) 173 pp. http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_500v5.pdf. (As of June 1, 2007).

17) Solomon, D., "Traffic Signals and Accidents in Michigan." *Public Roads*, Vol. 30, No. 10 (October 1959) pp. 234-237.

18) Cribbins, P.D., J.M. Arey, and J.K. Donaldson, "Effects of Selected Roadway and Operational Characteristics on Accidents on Multilane Highways." *Highway Research Record*, No. 188 (1967) pp. 8-25.

19) Cribbins, P.D., and C.M. Walton, "Traffic Signals and Overhead Flashers at Rural Intersections: Their Effectiveness in Reducing Accidents." *Highway Research Record*, No. 325 (1970) pp. 1-14.

20) Van Maren, P.A., *Correlation of Design and Control Characteristics with Accidents at Rural Multi-lane Highway Intersections in Indiana*, Purdue University and Indiana State Highway Commission Joint Highway Research Project, Indianapolis, Indiana (Revised July 1980) 60 pp.

21) Souleyrette, R.R., and T. Knox, *Safety Effectiveness of High-Speed Expressway Signals*, Center for Transportation Research and Education, Iowa State University, Ames, Iowa (August 2005) 23 pp. http://www.ctre.iastate.edu/reports/expressway_signals.pdf. (As of June 1, 2007).

22) Hauer, E., *Observational Before-After Studies in Road Safety: Estimating the Effect of Highway and Traffic Engineering Measures on Road Safety*, Elsevier Science, Inc., Kidlington, Oxford, UK (1997) 289 pp.

23) Khattak, A., L. Zhang, J.L. Hochstein, and S.C. Tee, "Crash Analysis of Expressway Intersections in Nebraska," *Transportation Research Board 85th Annual Meeting Compendium of Papers CD-ROM*, Washington, D.C. (January 2006).

24) Hanson, C., *Median Acceleration Lane Study Report*, Minnesota Department of Transportation District 6 Traffic Office, Rochester, MN (July 2002) 45 pp. http://www.dot.state.mn.us/trafficeng/research/data/MAL_final.pdf. (As of September 11, 2007).

25) Schurr, K.S., P.T. McCoy, G. Pesti, A.T. Egelhoff, and R. Burdick, *Deceleration Lanes on Left-Turn Bays of Four-Lane Expressways: Final Report*, Nebraska Department of Roads Project No. SPR-PL-1(038)P537, University of Nebraska-Lincoln, Lincoln, NE (December 2003) 149 pp.

26) Pant, P.D., and X.H. Huang, "Active Advance Warning Signs at High-Speed Signalized Intersections: Results of a Study in Ohio." *Transportation Research Record*, No. 1368 (1992) pp. 18-26.

27) Bonneson, J.A., P.T. McCoy, and J.E. Truby, Jr., "Safety Improvements at Intersections on Rural Expressways: A Survey of State Departments of Transportation." *Transportation Research Record*, No. 1385 (1993) pp. 41-47.

28) Potts, I.B., D.W. Harwood, D.J. Torbic, K.R. Richard, J.S. Gluck, H.S. Levinson, P.M. Garvey, and R.S. Ghebrial, *NCHRP Report 524: Safety of U-Turns at Unsignalized Median Openings*, TRB, Washington, D.C. (2004) 142 pp. http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_524.pdf. (As of September 17, 2007).

29) California Department of Transportation, *California Department of Transportation Highway Design Manual*, Sacramento, CA (2007). http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm#hdm. (As of October 22, 2007).

30) Reid, J., *Unconventional Arterial Intersection Design, Management and Operations Strategies*, Parsons Brinckerhoff, New York, NY (July 2004) 146 pp. http://www.pbworld.com/library/fellowship/reid/. (As of October 29, 2007).

31) Federal Highway Administration, *Manual on Uniform Traffic Control Devices for Streets and Highways - 2003 Edition with Revision 1 Incorporated*, U.S. Department of Transportation, Washington, D.C. (November 2004) http://mutcd.fhwa.dot.gov/kno-2003r1.htm. (As of June 1, 2007).

32) Agent, K.R., "Traffic Control and Accidents at Rural High-Speed Intersections (With Discussion and Closure)." *Transportation Research Record*, No. 1160 (1988) pp. 14-21.

33) Institute of Transportation Engineers, *ITE Speed Zoning Guidelines: Final Draft Version*, ITE Committee 4M-25, (undated). http://www.ibiblio.org/rdu/ite-szg.html. (As of November 2, 2007).

34) Staplin, L., "Simulator and Field Measures of Driver Age Differences in Left-Turn Gap Judgments." *Transportation Research Record*, No. 1485 (1995) pp. 49-55.

35) Harwood, D.W., and W.D. Glauz, *NCHRP Synthesis of Highway Practice 281: Operational Impacts of Median Width on Larger Vehicles*, TRB, National Research Council, Washington, D.C. (2000) 74 pp.

36) Kansas Department of Transportation, *Kansas Department of Transportation Design Manual, Volume 1 (Part A & B), Road Section*, Topeka, KS (November 2006) 810 pp. http://www.ksdot.org/kart/. (As of July 20, 2007).

37) Missouri Department of Transportation, *Missouri Department of Transportation Engineering Policy Guide*, Jefferson City, MO (2007). http://epg.modot.org/index.php?title=Main_Page. (As of July 20, 2007).

38) Staplin, L., K. Lococo, S. Byington, and D. Harkey, *Guidelines and Recommendations to Accommodate Older Drivers and Pedestrians*, Report No. FHWA-RD-01-051, Turner-Fairbank Highway Research Center, McLean, VA (May 2001) 90 pp. http://www.tfhrc.gov/humanfac/01105/cover.htm#toc. (As of August 17, 2007).

39) Stokes, R.W., M.J. Rys, E.R. Russell, R.K. Robinson, and B. Budke, *Analysis of Rural Intersection Accidents Caused By Stop Sign Violation and Failure to Yield the Right-of-Way*, Kansas Department of Transportation, Kansas State University, Manhattan, KS (September 2000) 44 pp. http://ntl.bts.gov/lib/10000/10800/10811/KSU-98-6.pdf. (As of November 14, 2007).

40) Hall, J.W., *Guidelines for the Installation of Intersection Beacons and Hazard Identification Beacons: Final Report*, New Mexico State Highway and Transportation Department, University of New Mexico, Albuquerque, NM (April 1991) 32 pp.

41) Minnesota Department of Transportation Office of Traffic, Safety, and Operations, *Minnesota Department of Transportation Traffic Engineering Manual*, Saint Paul, MN (August 2007). http://www.dot.state.mn.us/trafficeng/otepubl/tem/. (As of November 14, 2007).

42) Foody, T.J., and W.C. Taylor, "An Analysis of Flashing Systems." *Highway Research Record*, No. 221 (1968) pp. 72-84.

43) Hammer, J.B., and E.J. Tye, *Overhead Yellow-Red Flashing Beacons: Final Report*, Federal Highway Administration, California Department of Transportation, Sacramento, CA (January 1987) 53 pp.

44) Pant, P.D., Y. Park, and S.V. Neti, *Development of Guidelines for Installation of Intersection Control Beacons: Final Report*, Ohio Department of Transportation, University of Cincinnati, Cincinnati, OH (December 1992) 130 pp.

45) Preston, H., and T. Schoenecker, *Safety Impacts of Street Lighting at Isolated Rural Intersections: Final Report*, Minnesota Department of Transportation, St. Paul, MN (April 1999) 74 pp. http://www.lrrb.org/pdf/199917.pdf. (As of November 28, 2007).

46) Isebrands, H.N., S. Hallmark, Z. Hans, T. McDonald, H. Preston, and R. Storm, *Safety Impacts of Street Lighting at Isolated Rural Intersections – Part II: Final Report*, Minnesota Department of Transportation, St. Paul, MN (June 2006) 120 pp. http://www.lrrb.org/pdf/200635.pdf. (As of November 28, 2007).

47) Corkle, J., M. Marti, and D. Montebello, *Synthesis on the Effectiveness of Rumble Strips*, Minnesota Department of Transportation, St. Paul, MN (October 2001) 55 pp. http://www.lrrb.org/pdf/200207.pdf. (As of November 29, 2007).

48) Harder, K.A., J. Bloomfield, and B. Chihak, *The Effects of In-Lane Rumble Strips on the Stopping Behavior of Attentive Drivers*, Minnesota Department of Transportation, St. Paul, MN (October 2001) 28 pp. http://www.lrrb.org/pdf/200211.pdf. (As of November 29, 2007).

49) Harwood, D.W., *NCHRP Synthesis of Highway Practice 191: Use of Rumble Strips to Enhance Safety*, TRB, National Research Council, Washington, D.C. (1993) 80 pp.

50) Preston, H., R. Storm, M. Donath, and C. Shankwitz, *Review of North Carolina's Rural Intersection Crashes: Application of Methodology for Identifying Intersections for Intersection Decision Support (IDS), Report #2 in the Series: Toward a Multi-State Consensus on Rural Intersection Decision Support*, Minnesota Department of Transportation, St. Paul, MN (August 2006) 56 pp. http://www.lrrb.org/pdf/200632.pdf. (As of November 30, 2007).

51) Sisiopiku, V.P., and L. Aylsworth-Bonzelet, "*Application of Loons at Directional Crossovers*" 82nd Annual Meeting of the Transportation Research Board, Washington, D.C., Proceedings CD-ROM (2003) pp. 1-14. http://www.ltrc.lsu.edu/TRB_82/TRB2003-001541.pdf. (As of December 21, 2007).

52) Liu, P., J. Lu, H. Chen, and G. Sokolow, "*Understanding Factors Affecting Safety of Right-Turns Followed by U-Turns*" 87th Annual Meeting of the Transportation Research Board, Washington, D.C., Proceedings CD-ROM (2008) pp. 1-14.

53) North Carolina Department of Transportation, *North Carolina Department of Transportation Roadway Design Manual*, Raleigh, NC (2007). http://www.ncdot.org/doh/preconstruct/altern/value/manuals/designmanual.html. (As of July 20, 2007).

54) Florida Department of Transportation, *Design Standards for Design, Construction, Maintenance, and Utility Operations on the State Highway System*, Tallahassee, FL (2006). http://www.dot.state.fl.us/rddesign/rd/RTDS/06/2006Standards.htm. (As of July 20, 2007).

55) Maryland State Highway Administration, Conceptual Drawing of J-Turn Intersection at US-301 & MD-313, District 2 Office, Chestertown, MD.

56) Maryland State Highway Administration, *Maryland's Traffic Volume Maps by County*. http://www.sha.state.md.us/SHAServices/mapsBrochures/maps/OPPE/tvmaps.asp. (As of February 7, 2008).

57) Maryland State Highway Administration, Collision Diagrams at US-301 & MD-313, Office of Traffic & Safety, Crash Analysis Safety Team, 5/30/2007.

58) North Carolina Department of Transportation, Safety Evaluation Group Homepage. http://www.ncdot.org/doh/preconstruct/traffic/safety/TSSM/se.html. (As of February 13, 2008).

59) North Carolina Department of Transportation Safety Evaluation Group, *Spot Safety Project Evaluation of the Directional Crossover Installation at the Intersection of US-23/74 at SR-1527 (Steeple Drive) and SR-1449 (Cope Creek Road) Near Sylva, Jackson County*, NCDOT, Raleigh, NC (September 14, 2005) 12 pp. http://www.ncdot.org/doh/preconstruct/traffic/Safety/ses/projects/completed_files/docs/SS1497017.pdf. (As of February 20, 2008).

60) North Carolina Department of Transportation Safety Evaluation Group, *Spot Safety Project Evaluation of the Directional Crossover Installation at the Intersection of US-64 and SR-2234/SR-2500 (Mark's Creek Road) Near Knightdale in Wake County*, NCDOT, Raleigh, NC (September 16, 2005) 12 pp. http://www.ncdot.org/doh/preconstruct/traffic/Safety/ses/projects/completed_files/docs/SS0599243.pdf. (As of February 20, 2008).

61) North Carolina Department of Transportation Safety Evaluation Group, *Spot Safety Project Evaluation of the Median Directional Crossover Installation and the Left-Turn Storage Lanes Extension at the Intersection of US-321 (Hickory Boulevard) and SR-1796 (Victoria Court/Clover Drive) in Caldwell County*, NCDOT, Raleigh, NC (February 8, 2006) 13 pp. http://www.ncdot.org/doh/preconstruct/traffic/Safety/ses/projects/completed_files/docs/SS1199210.pdf. (As of February 20, 2008).

62) Bared, J.G., and E.I. Kaisar, "Advantages of Offset T-Intersections with Guidelines" International Conference: Traffic Safety on Three Continents, Moscow, Russia, Proceedings CD-ROM (November 2001) 16p. http://www.vti.se/EPiBrowser/Publikationer/K18APart2.pdf (As of April 9, 2008).

63) Mahalel, D., J. Craus, and A. Polus, "Evaluation of Staggered and Cross Intersections." *Journal of Transportation Engineering*, Vol. 112, No. 5 (September 1986) pp. 495-506.

64) Institute of Transportation Engineers, *Toolbox on Intersection Safety and Design*, U.S. Department of Transportation, FHWA, Washington, D.C. (September 2004) 170 pp.

65) Hummer, J.E., and J.L. Boone, "Travel Efficiency of Unconventional Suburban Arterial Intersection Designs." *Transportation Research Record*, No. 1500 (1995) pp. 153-161.

66) Hummer, J.E., "Unconventional Left-Turn Alternatives for Urban and Suburban Arterials – Part Two." *ITE Journal on the Web* (November 1998) pp. 101-106.

67) Institute of Transportation Engineers, *The Traffic Safety Toolbox: A Primer on Traffic Safety (2nd Edition)*, ITE, Washington, D.C. (1999) 317 pp.

68) New Jersey Department of Transportation, *New Jersey Department of Transportation Roadway Design Manual*, Trenton, NJ. http://www.state.nj.us/transportation/eng/documents/RDME/index.shtml. (As of July 13, 2007).

69) Jagannathan, R., M.A. Gimbel, J.G. Bared, W.E. Hughes, B. Persuad, and C. Lyon, "*Safety Comparison of New Jersey Jughandle Intersections and Conventional Intersections*" 85th Annual Meeting of the Transportation Research Board, Washington, D.C., Proceedings CD-ROM (2006) pp. 1-28.

70) Staplin, L., D.L. Harkey, K.H. Lococo, and M.S. Tarawneh, *Intersection Geometric Design and Operational Guidelines for Older Drivers and Pedestrians, Volume I: Final Report*, Publication No. FHWA-RD-96-132, Federal Highway Administration, McLean, VA (June 1997) 249 pp.

71) Rodegerdts, L.A., B. Nevers, B. Robinson, J. Ringert, P. Koonce, J. Bansen, T. Nguyen, J. McGill, D. Stewart, J. Suggett, T. Neuman, N. Antonucci, K. Hardy, and K. Courage, *Signalized Intersections: Informational Guide*, Publication No. FHWA-HRT-04-091, Federal Highway Administration, McLean, VA (August 2004) 369 pp. www.tfhrc.gov/safety/pubs/04091/04091.pdf (As of May 5, 2008).

72) Jagannathan, R., *Traffic Performance of Three Typical Designs of New Jersey Jughandle Intersections*, Publication No. FHWA-HRT-07-032, Federal Highway Administration, McLean, VA (2007) 12 pp. www.tfhrc.gov/safety/pubs/07032/index.htm (As of May 5, 2008).

73) Pline, J.L., *NCHRP Synthesis of Highway Practice 225: Left-Turn Treatments at Intersections*, TRB, National Research Council, Washington, D.C. (1996) 92 pp.

74) Najm, W.G., J.A. Koopmann, and D.L. Smith, "*Analysis of Crossing Path Crash Countermeasure Systems*" Proceedings of the 17th International Technical Conference on the Enhanced Safety of Vehicles (Paper #378), Amsterdam, the Netherlands (June 2001) pp. 1-9. http://www-nrd.nhtsa.dot.gov/pdf/nrd-01/Esv/esv17/Proceed/00013.pdf (As of May 8, 2008).

75) Chovan, J.D., L. Tijerina, J.A. Pierowicz, and D.L. Hendricks, *Examination of Unsignalized Intersection, Straight Crossing Path Crashes and Potential IVHS Countermeasures: Final Report*, Volpe National Transportation Systems Center, Cambridge, MA (August 1994) 72 pp. http://www.itsdocs.fhwa.dot.gov/JPODOCS/REPTS_TE/7824.pdf (As of May 8, 2008).

76) Harder, K.A., J. Bloomfield, and B.J. Chihak, *Reducing Crashes at Controlled Rural Intersections*, University of Minnesota, Minnesota Department of Transportation, St. Paul, MN (July 2003) 49 pp. http://www.lrrb.org/pdf/200315.pdf (As of May 8, 2008).

77) Creaser, J., M. Rakauskas, N. Ward, and J. Laberge, *A Simulator-Based Evaluation of Smart Infrastructure Concepts for Intersection Decision Support for Rural Thru-STOP Intersections: Report #4 in the Series: Developing Intersection Decision Support Solutions*, University of Minnesota, Minnesota Department of Transportation, St. Paul, MN (August 2007) 301 pp. http://www.lrrb.org/PDF/200731.pdf (As of May 8, 2008).

78) Rural Intersection Collision Avoidance: Toward a Multi-State Consensus on Rural Intersection Decision Support Project Homepage. http://www.its.umn.edu/Research/FeaturedStudies/IDS/pooledfund/index.html (As of May 8, 2008).

79) Hanscom, F.R., *Evaluation of the Prince William County Collision Countermeasure System*, Report No. FHWA/VTRC 01-CR5, Virginia Transportation Research Council, Charlottesville, VA (February 2001) 26 pp. http://ntl.bts.gov/lib/17000/17600/17698/PB2001103190.pdf. (As of August 15, 2007).

80) Maine Department of Transportation – Transportation Research Division, *Evaluation of the Norridgewock Intersection Collision Avoidance Warning System on Route 201A, Norridgewock, Maine*, Final Technical Report #01-2, Maine Department of Transportation, Augusta, ME (November 2006) 18 pp. http://www.maine.gov/mdot/transportation-research/pdf/Report0102f.pdf. (As of August 15, 2007).

81) Research and Innovative Technology Administration (RITA) ITS Homepage. http://www.its.dot.gov/itsnews/fact_sheets/cicas.htm (As of April 23, 2008).

82) Zwahlen, H.T., E. Oner, F.F. Badurdeen, and A. Russ, *Human Factors Opportunities to Improve Ohio's Transportation System*, Ohio University, Ohio Department of Transportation, Columbus, OH (June 2005) 299 pp. http://www.dot.state.oh.us/research/2005/Traffic/134154-FR.pdf (As of May 9, 2008).

83) Institute of Transportation Engineers Technical Committee 5B-4, "Effectiveness of Median Storage and Acceleration Lanes for Left-Turning Vehicles." *ITE Journal*, Vol. 55, No. 3 (March 1985) pp. 20-25.

84) Minnesota Department of Transportation, *Minnesota Department of Transportation Road Design Manual*, Saint Paul, MN (June 2000). http://www.dot.state.mn.us/tecsup/rdm/. (As of July 20, 2007).

85) Iowa Department of Transportation, *Iowa Department of Transportation Design Manual*, Ames, IA (May 2007). ftp://165.206.203.34/design/dmanual/00_START%20HERE_TOC.pdf. (As of July 20, 2007).

86) Zeidan, G.R., and P.T. McCoy, "Effects of Right-Turn Lanes on Driveway Sight Distance." *Transportation Research Record*, No. 1737 (2000) pp. 78-83.

87) Iowa Department of Transportation, *Iowa's City and County Annual Average Daily Traffic Maps*. http://www.iowadotmaps.com/msp/traffic/index.html. (As of May 27, 2008).

88) Neuman, T.R., *NCHRP Report 279: Intersection Channelization Design Guide*, Transportation Research Board, Washington, D.C. (November 1985) 153 pp.

89) Transportation Research Board Research in Progress (RiP) Online Database. http://rip.trb.org/browse/dproject.asp?n=11936. (As of May 31, 2008).

90) McCoy, P.T., U.R. Navarro, and W.E. Witt, "Guidelines for Offsetting Opposing Left-Turn Lanes on Four-Lane Divided Roadways." *Transportation Research Record*, No. 1356 (1992) pp. 28-36.

91) Illinois Department of Transportation, *Bureau of Design & Environment Manual – 2002 Edition*, Springfield, IL (December 2002). http://www.dot.state.il.us/desenv/bdemanual.html. (As of July 11, 2007).

92) North Carolina Department of Transportation Safety Evaluation Group, *Spot Safety Project Evaluation of the Offset Left-Turn Lane Installation at the Intersection of US-421 (Carolina Beach Road) and SR-1576/1531 (River Road/Seabreeze Road)*, NCDOT, Raleigh, NC (June 2006) 11 pp. http://www.ncdot.org/doh/preconstruct/traffic/Safety/ses/projects/completed_files/docs/SS0399204.pdf. (As of June 20, 2008).

93) North Carolina Department of Transportation Safety Evaluation Group, *Spot Safety Project Evaluation of the Offset Left-Turn Lane Installation at the Intersection of US-421 (Carolina Beach Road) and SR-1524 (Golden Road)/Masonboro Commons Shopping Center*, NCDOT, Raleigh, NC (June 2006) 11 pp. http://www.ncdot.org/doh/preconstruct/traffic/Safety/ses/projects/completed_files/docs/SS0301213.pdf. (As of June 20, 2008).

94) Preston, H., and C. Rasmussen, *T.H. 52 (Rochester to Inver Grove Heights) Road Safety Audit*, Minnesota Department of Transportation and the Department of Public Safety, St. Paul, MN (February 2003).

95) North Carolina Department of Transportation Safety Evaluation Group, *Spot Safety Project Evaluation of the Post Mounted "Vehicle Entering When Flashing" Signs at the Intersection of US-74/76 and SR-1800 (Blacksmith Road) Near Bolton, Columbus County*, NCDOT, Raleigh, NC (August 29, 2005) 10 pp. http://www.ncdot.org/doh/preconstruct/traffic/Safety/ses/projects/completed_files/docs/SS0695402.pdf. (As of June 20, 2008).

96) North Carolina Department of Transportation Safety Evaluation Group, *Spot Safety Project Evaluation of the Post Mounted "Vehicle Entering When Flashing" Signs at the Intersection of US-421 and NC-210 Near Currie, Pender County*, NCDOT, Raleigh, NC (March 24, 2005) 10 pp. http://www.ncdot.org/doh/preconstruct/traffic/Safety/ses/projects/completed_files/docs/SS0396203.pdf. (As of June 20, 2008).

97) Leisch, J.P., *Institute of Transportation Engineers Freeway and Interchange Geometric Design Handbook*, ITE, Washington, D.C. (2005) 639 pp.