

System-Wide Safety Treatments and Design Guidance for J-Turns

Final Report
January 2017

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Technical Report Documentation Page

1. Report No. MoDOT cmr 16-013	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle System-Wide Safety Treatments and Design Guidance for J-Turns		5. Report Date January 2017	
		6. Performing Organization Code	
7. Author(s) Praveen Edara, Carlos Sun (http://orcid.org/0000-0002-8857-9648), Boris Claros, Zhongyuan Zhu, and Henry Brown		8. Performing Organization Report No.	
9. Performing Organization Name and Address Civil & Environmental Engineering University of Missouri-Columbia E2509 Lafferre Hall Columbia, MO 65211		10. Work Unit No. (TRAIS)	
		11. Contract or Grant No. MoDOT project # TR201510 Part of USDOT DTRT13-G-UTC37	
12. Sponsoring Organization Name and Address <div style="display: flex; justify-content: space-between;"> <div style="width: 45%;"> Midwest Transportation Center 2711 S. Loop Drive, Suite 4700 Ames, IA 50010-8664 U.S. DOT Office of the Assistant Secretary for Research and Technology 1200 New Jersey Avenue, SE Washington, DC 20590 </div> <div style="width: 45%;"> Missouri Department of Transportation (SPR) http://dx.doi.org/10.13039/100007251 Construction & Materials Division P.O. Box 270 Jefferson City, MO 65102 </div> </div>		13. Type of Report and Period Covered Final Report for July 2014–April 2016	
		14. Sponsoring Agency Code	
15. Supplementary Notes Visit http://www.intrans.iastate.edu/ for color pdfs of this and other research reports. MoDOT research reports are available in the Innovation Library at http://www.modot.org/services/or/byDate.htm . This report is also available at https://library.modot.mo.gov/RDT/reports/TR201510/cmr16-013.pdf .			
16. Abstract <p>In support of its Toward Zero Deaths (TZD) initiative, the Missouri Department of Transportation (MoDOT) initiated this research project to develop guidance on treatments that can reduce crashes and fatalities. The first task for this project was to synthesize the literature and state of the practice related to system-wide safety treatments and document the treatments' effectiveness. In particular, the objective was to examine those treatments that have not already been implemented in Missouri. The safety effectiveness, implementation guidelines, limitations, costs, and concerns of the treatments were documented. The identified safety treatments work in conjunction with the "Necessary Nine" strategies identified in the Missouri Blueprint. Accordingly, the synthesis covered three areas: (1) horizontal curves, (2) intersections, and 3) wrong-way crashes. The reviewed treatments included signing, geometric design and access management, intelligent transportation systems (ITS), pavement markings, and signal control enhancements to improve safety. In the last few years, MoDOT has replaced several high-crash intersections on rural highways in the state with J-turns. Given their safety effectiveness and low cost, J-turns have become a preferred alternative to replace high-crash two-way stop-controlled intersections on high-speed highways. Unfortunately, national guidance on the design of J-turns is very limited. This project addressed this gap in practice by developing guidance on spacing and acceleration lanes. A thorough examination of crashes that occurred at 12 existing J-turn sites in Missouri was conducted. The crash review revealed the proportions of five crash types occurring at J-turn sites: (1) major road sideswipe (31.6%), (2) major road rear-end (28.1%), (3) minor road rear-end (15.8%), (4) loss of control (14%), and (5) merging from U-turn (10.5%). The crash rates for both sideswipe and rear-end crashes decreased with an increase in the spacing from the minor road to the U-turn; J-turns with a spacing of 1,500 ft or greater experienced the lowest crash rates. A calibrated simulation model was used to study various volume scenarios and design variables. For all scenarios, the presence of an acceleration lane resulted in significantly fewer conflicts. Therefore, acceleration lanes are recommended for all J-turn designs, including lower volume sites. Moreover, while a spacing between 1,000 and 2,000 ft was found to be sufficient for low-volume combinations, a spacing of 2,000 ft is recommended for medium- to high-volume conditions.</p>			
17. Key Words design guidance—guidelines—highway design—highway safety—intersections—J-turns—system-wide safety—turning traffic—U-turns		18. Distribution Statement No restrictions. This document is available through the National Technical Information Service, Springfield, VA 22161.	
19. Security Classification (of this report) Unclassified.	20. Security Classification (of this page) Unclassified.	21. No. of Pages 98	22. Price NA

SYSTEM-WIDE SAFETY TREATMENTS AND DESIGN GUIDANCE FOR J-TURNS

**Final Report
January 2017**

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Sponsored by
Missouri Department of Transportation,
Midwest Transportation Center, and
U.S. Department of Transportation
Office of the Assistant Secretary for Research and Technology

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ACKNOWLEDGMENTS

This project was funded by the Missouri Department of Transportation (MoDOT), the Midwest Transportation Center, and the U.S. Department of Transportation Office of the Assistant Secretary for Research and Technology.

The authors acknowledge the assistance provided by John Miller, Mike Curtit, Andrew Williford, Jon Nelson, Myrna Tucker, Darrell Knierim, Jason Collins, Jen Harper, and William Stone from MoDOT.

EXECUTIVE SUMMARY

The Missouri Department of Transportation (MoDOT) sponsored this research project to investigate treatments that can reduce crashes and fatalities to further the goal of the Toward Zero Deaths (TZD) initiative. One major objective was to synthesize the literature and state of the practice related to system-wide safety treatments and document the treatments' effectiveness. Specifically, the objective was to examine those treatments that have not already been implemented in Missouri. Another major objective was to provide guidance on the design of the J-turn intersection, which eliminates or reduces crossing conflicts.

A synthesis of system-wide safety treatments from other states and countries was conducted. The safety effectiveness, implementation guidelines, limitations, costs, and concerns of the treatments were documented. The identified safety treatments are consistent with the "Necessary Nine" strategies identified in the Missouri Blueprint. Accordingly, the synthesis covered three areas: (1) horizontal curves, (2) intersections, and (3) wrong-way crashes. The reviewed treatments include signing, geometric design and access management, intelligent transportation systems (ITS), pavement markings, and signal control enhancements to improve safety. This synthesis provides a systematic method for selecting system-wide treatments for future deployments in the state of Missouri.

Countermeasures related to signage, design, ITS, and drivers were reviewed to address wrong-way crashes. Innovative signage strategies including lowering the height of signs, deploying oversized signs, providing illumination, and doubling the number of signs are low-cost solutions that can be deployed across the system. Design countermeasures such as avoiding left-side exit ramps, using raised medians on crossroads, and improving sight distance are also recommended. ITS technology options, due to their higher costs, may not be suitable for system-wide deployment but are appropriate for isolated treatments. Detection and alert systems based on video radar or in-pavement sensors have been piloted in a few states.

Countermeasures targeting horizontal curve crashes may include signage treatments that exceed the minimum signage and device requirements recommended by the *Manual on Uniform Traffic Control Devices* (MUTCD) for horizontal curves. Such treatments include improved curve signing through the use of additional chevrons, flashing beacons at sharp curves, dynamic curve guidance systems, and dynamic speed warning systems. Other recommended horizontal curve safety treatments include pavement treatments such as speed reduction markings, warning symbols painted on the pavement, and high-friction pavement treatments. MoDOT has successfully utilized two pavement marking treatments in the past: wider edge lines and rumble strips/stripes.

Treatments to enhance signalized intersection safety include increasing clearance intervals, changing left turns from permissive to protected-permissive, installing flashing yellow arrows, providing dynamic signal warning, installing red light cameras, and improving signal visibility. Based on the safety effectiveness reported in literature, providing dynamic signal warning and improving signal visibility are recommended for future consideration as system-wide treatments at signalized intersections in Missouri. At stop-controlled intersections, the use of bigger signs,

LEDs, and flashing beacons was found to reduce crashes due to the increased visibility and illumination of signs.

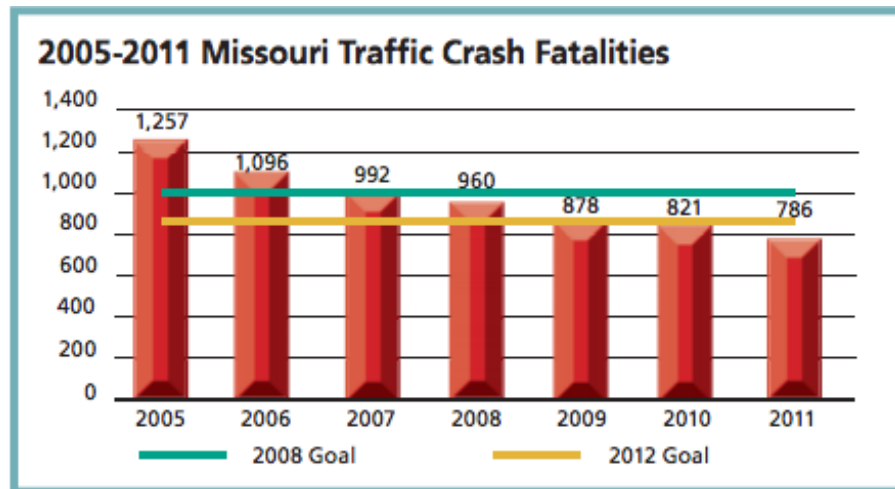
In the last few years, MoDOT has replaced several high-crash intersections on rural highways in the state with J-turns. Given their safety effectiveness and low cost, J-turns have become a preferred alternative to grade-separated interchanges for replacing high-crash two-way stop-controlled intersections on high-speed highways. Unfortunately, national guidance on the design of J-turns is very limited. For example, there are no recommendations on the spacing between the main intersection and the U-turn. Similarly, there is no guidance on when acceleration lanes are recommended, i.e., at what level of traffic volume. This project addressed this gap in practice by developing guidance on spacing and acceleration lanes. A thorough examination of crashes that occurred at 12 existing J-turn sites in Missouri was conducted. The objective of this review was to determine if the crash frequencies and types of crashes were influenced by the aforementioned design parameters.

The crash review revealed the proportions of five crash types occurring at J-turn sites: (1) major road sideswipe (31.6%), (2) major road rear-end (28.1%), (3) minor road rear-end (15.8%), (4) loss of control (14%), and (5) merging from U-turn (10.5%). Vehicles merging with the major road traffic or changing lanes to access the U-turn lane caused most of the major road sideswipe and rear-end crashes. Other common contributing factors included driver inattention and the large speed difference between the merging vehicles from the minor road and the vehicles on the major road. Crash rates, expressed as per million vehicle miles of travel, decreased with an increase in the U-turn spacing for both sideswipe and rear-end crashes. A longer spacing allowed merging vehicles to reach major road operating speeds, thus making it safer to follow other vehicles in the lane and to make lane changes. J-turn sites with a spacing of 1,500 ft or greater experienced the lowest crash rates.

In addition, traffic simulation experiments were conducted to study the effect of different design parameters and traffic volumes on the safety of the J-turn design. A base simulation model was created and calibrated using field data collected during a previous MoDOT project on J-turns. The calibrated model was then used to study various combinations of major road and minor road volumes and design variables. For all of the studied scenarios, the presence of an acceleration lane resulted in significantly fewer conflicts. Therefore, acceleration lanes are recommended for all J-turn designs, including those at lower volume sites. Second, while a spacing between 1,000 and 2,000 ft was found to be sufficient for low-volume combinations, a spacing of 2000 ft is recommended for medium- to high-volume conditions.

1. BACKGROUND

Traffic fatalities in Missouri have decreased steadily in the last decade. Figure 1.1 shows the trend in road fatalities since 2005, as reported in the *Missouri's Blueprint to Save More Lives 2012–2016* (MCRS 2012).



Missouri Coalition for Roadway Safety (MCRS) 2012

Figure 1.1. Traffic fatalities in Missouri (2005–2011)

A major factor that has led to this reduction in fatalities is that the Missouri Department of Transportation (MoDOT) has targeted specific crash types with system-wide safety treatments. This approach has been shown to be more effective than spot improvements due to the inherent randomness of crash occurrence locations on road segments. In the last decade, MoDOT has implemented system-wide safety treatments, such as median cable barriers (see Figure 1.2) and rumble strips, that have produced significant safety results.



MoDOT

Figure 1.2. Median cable barrier

For example, it is estimated that the 800 miles of cable median barriers installed in Missouri have resulted in at least 300 lives saved in over a decade. Missouri's Blueprint establishes a short-term goal of reducing traffic fatalities to 700 per year by 2016. This goal is geared towards achieving the long-term vision of zero roadway deaths in the state.

MoDOT initiated this research project to accomplish two major objectives. The first objective is to synthesize existing practices on system-wide safety treatments, especially those treatments that have not been implemented already in Missouri. The second objective is to develop design guidance for J-turns, which are being increasingly adopted across Missouri. J-turns are an effective and low-cost safety treatment, especially at rural high-speed expressway intersections. Taken together, these two objectives will assist MoDOT in decreasing crashes and saving lives in Missouri.

1.1 Goal 1: Synthesis of System-wide Safety Treatments

A synthesis of system-wide safety treatments from other states and countries was conducted. The safety effectiveness, implementation guidelines, limitations, costs, and concerns of the treatments were documented. The identified safety treatments work in conjunction with the "Necessary Nine" strategies identified in the Missouri Blueprint (MCRS 2012). The necessary nine strategies were identified as the strategies with the greatest potential to save lives and reduce serious injuries. They include strategies to (1) increase safety belt use, (2) expand the installation of rumble strips/stripes, (3) increase efforts to reduce the number of impaired drivers, (4) improve intersection safety, (5) improve curve safety, (6) change traffic safety culture, (7) improve roadway shoulders, (8) increase enforcement efforts, and (9) expand and improve roadway visibility. System-wide safety treatments that address these strategies will be of immediate value to transportation agencies in Missouri and can be implemented in the near future.

Similarly, the identified treatments are associated with the Blueprint's areas of emphasis and focus. For example, the "serious crash types" emphasis area focuses on reducing horizontal curve crashes and intersection crashes, among others. According to the Missouri Blueprint, "A driver is three times more likely to be involved in a crash on a horizontal curve than on a straight stretch of roadway. In Missouri, 33.2 percent of all fatalities and 27 percent of all serious injuries during the past three years occurred along horizontal curves" (MCRS 2012).

The focus of the synthesis was on treatments that have not been implemented previously in Missouri. For example, literature on rumble strips/stripes was not considered to be of high importance to MoDOT because these treatments have already been deployed on several highways across the state. Accordingly, the review was broadly grouped into treatments applicable to three areas: (1) horizontal curves, (2) intersections, and (3) wrong-way crashes. These three areas were also recommended by the project's technical advisory panel. The numbers of fatalities and serious injuries that occurred in Missouri between 2009 and 2011, including those that occurred on horizontal curves and intersections, are presented in Table 1.1.

Table 1.1. Fatality and serious injury statistics in Missouri, 2009 to 2011

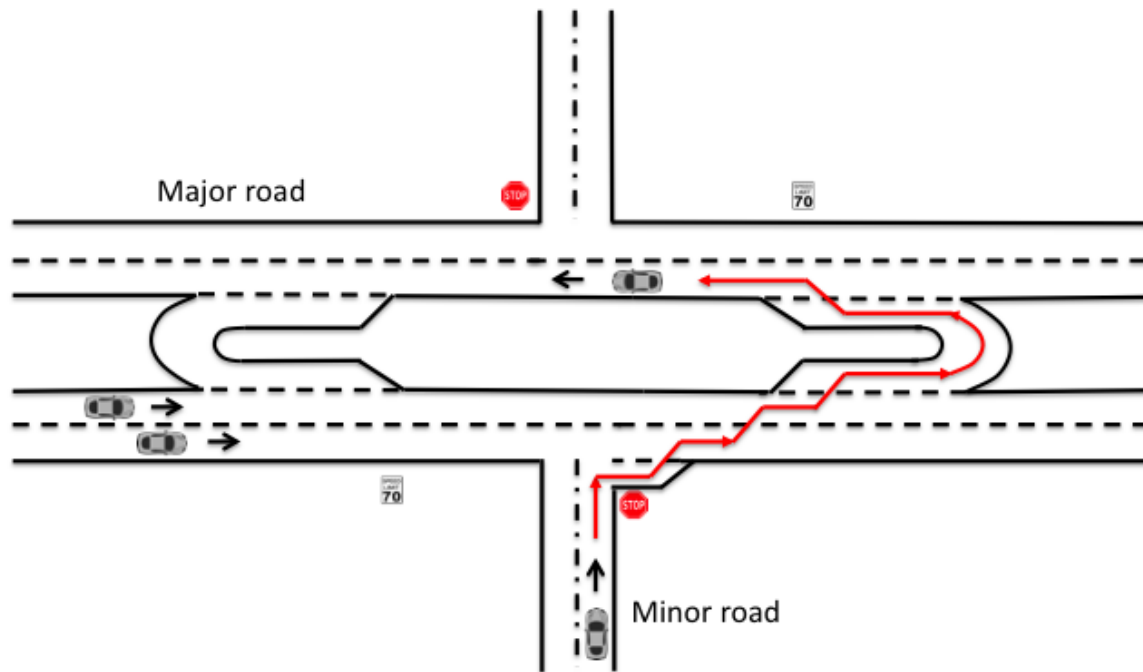
Crash Location/Type	2009		2010		2011	
	Fatalities	Serious Injuries	Fatalities	Serious Injuries	Fatalities	Serious Injuries
Missouri Total	878	6540	821	6096	786	5644
Horizontal Curves	293	1783	262	1636	270	1521
Intersections	150	1926	165	1747	113	1642
Head-on	140	582	106	478	121	487

Source: MCRS 2012

The three areas, horizontal curves, intersections, and head-on crashes, accounted for more than 65% of fatalities in Missouri from 2009 to 2011. The last row of Table 1.1 shows head-on crashes, the majority of which occurred on two-lane highways due to vehicles crossing the centerline and colliding with oncoming traffic. Countermeasures such as centerline rumble stripes are already used by MoDOT on several two-lane highways across the state to alert drivers of lane departures. Another cause of head-on crashes is wrong-way driving. MoDOT is currently placing an emphasis on mitigating wrong-way crashes across the state, which has included pilot deployments of treatments in the St. Louis region.

1.2 Goal 2: Design Guidance for J-turns

At a traditional two-way stop-controlled (TWSC) intersection on a four-lane divided highway, vehicles accessing the major highway from the minor road can make a left turn or through movement at the intersection by crossing major road movements. Highways with high volumes and/or high speeds may make these minor road movements challenging to execute. In contrast, in a J-turn design, vehicles accessing the major highway from the minor road make a right turning movement and then use a U-turn at a downstream location. The major road vehicles accessing the minor road via a left turning movement may or may not have to use the U-turn for their movements. One variation of the J-turn design allows for major road turning movements to occur at the intersection but still requires the minor road movements to use the U-turn. A conceptual schematic of the J-turn intersection is shown in Figure 1.3.



Not to scale

Figure 1.3. Conceptual schematic of J-turn intersection

In Figure 1.3, the left turning movement from the minor road is shown using red arrows. The safety of the J-turn design stems from the reduction of severe high-risk conflict points, including crossing conflicts, which result in right-angle crashes. According to NCHRP 650 (Maze et al. 2010), a TWSC intersection on a four-lane divided highway has 42 conflict points, while a J-turn intersection has 24 conflict points.

MoDOT has replaced several high-crash intersections on rural highways in the state with J-turns. A recent study (Edara et al. 2014) quantified the overall safety benefits of J-turns in Missouri. Given their safety effectiveness and low cost compared to grade-separated interchanges, the J-turn has become a preferred replacement for high-crash TWSC locations on high-speed highways. The J-turn design has been in use in the U.S. for several years under other names, such as “superstreet” in North Carolina and “restricted crossing U-turn” in Maryland. Despite their long use, there is no specific national guidance on the design of J-turns. For example, there are no recommendations on the spacing between the main intersection and the U-turn. Similarly, there is no guidance regarding when acceleration lanes are recommended, i.e., at what level of traffic volume. To this end, this project used a two-pronged approach to develop guidance for designing J-turns. First, a thorough review of crashes that occurred at existing J-turn sites in Missouri was conducted. The objective of this review was to identify how the crash frequencies and types were influenced by any design parameters. Second, traffic simulation experiments were conducted to study the effects of different design parameters and traffic volumes. The simulation experiments measured the safety effects of the presence of an acceleration lane and the spacing between the main intersection and the U-turn.

Various combinations of minor road and major road volumes were analyzed for different spacing values. Vehicle trajectories were extracted from the simulation. The vehicle trajectories provided information about the longitudinal and lateral location of vehicles, speed, acceleration, and other characteristics at every 0.1 seconds. The vehicle trajectory data were used to extract conflict safety measures such as the time to conflict (TTC), post-encroachment time, and conflict angle, which were in turn used to quantify the number of lane change conflicts. Recall that crossing conflicts resulting from minor road left turns at TWSCs are replaced at J-turns by lane change conflicts. The Federal Highway Administration's (FHWA's) Surrogate Safety Assessment Model (SSAM), used in previous studies to generate conflict measures from simulation models (Gettman and Head 2003, Kim et al. 2007), produced the aforementioned safety performance measures.

2. SYNTHESIS OF WRONG-WAY CRASHES

Wrong-way driving is a rare but dangerous event that can lead to severe crashes. Existing research on wrong-way driving crashes has focused on contributing factors and countermeasures to mitigate them. Contributing factors include driver, vehicle, and facility characteristics. A synthesis of the contributing factors and countermeasures is presented in this chapter.

2.1 Contributing Factors

2.1.1 Age

Drivers over the age of 70 and young drivers are overrepresented as at-fault drivers in wrong-way crashes (Braam 2006). Most of the crashes caused by young drivers were due to inattention, while most crashes caused by older drivers occurred because of some physical illnesses such as dementia or confusion (Braam 2006, Kemel 2015, Zhou et al. 2012). Figure 2.1 shows a comparison of the ages of wrong-way drivers with the ages of the drivers of other vehicles involved in fatal wrong-way collisions. The age distribution of the drivers of the other vehicles involved (i.e., the right-way drivers) represents a typical age distribution in such crashes.

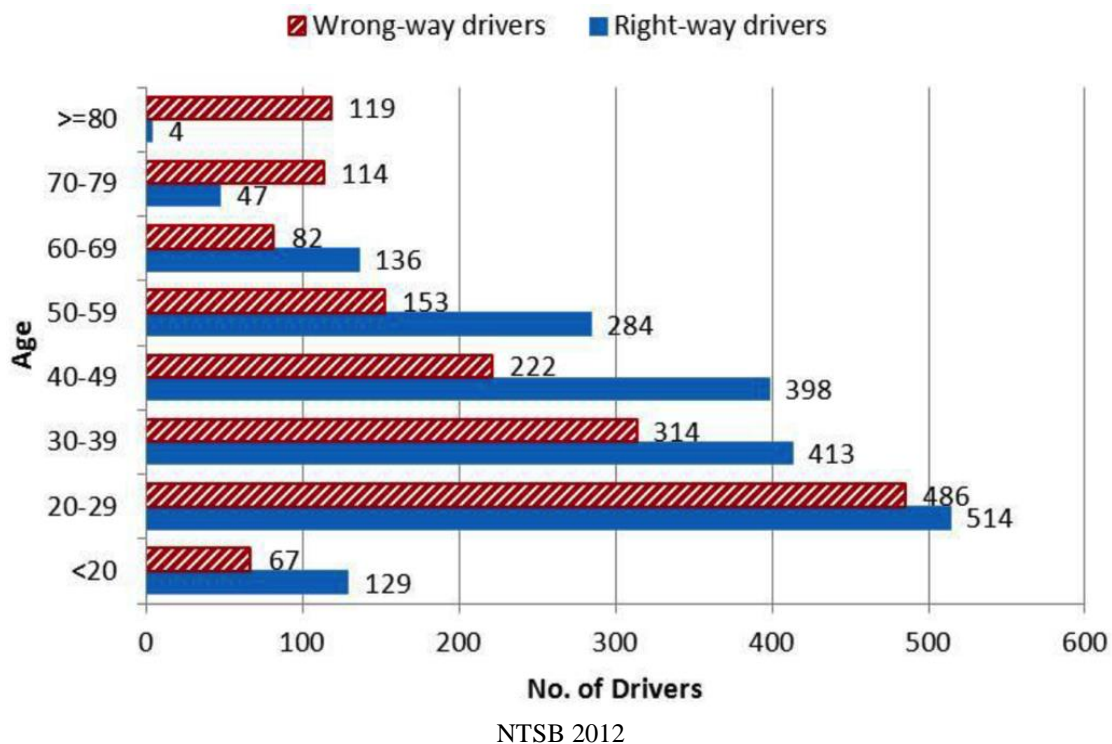


Figure 2.1. Comparison between wrong-way and right-way driver ages in fatal wrong-way collisions

2.1.2 Gender

Male drivers are overrepresented in wrong-way crashes. In a study conducted in Texas, 67% of wrong-way crashes involved male drivers. This overrepresentation has also been found outside the U.S.: 76% of wrong-way crashes in France and 81% in Holland involved male drivers (Kemel 2015, Zhou et al. 2012, Cooner et al. 2004, SWOV 2012).

2.1.3 Impaired Driving

A recent study performed by the National Transportation Safety Board (NTSB) using Fatality Analysis Reporting System (FARS) data from the National Highway Traffic Safety Administration (NHTSA) found that 60% of wrong-way crashes involved drivers impaired by alcohol and another 3.4% involved drivers who had been drinking without going over the legal alcohol limit (NTSB 2012). The NTSB (2012) study also reported that evidence of drug use was found in 4.4% of impaired drivers involved in wrong-way crashes.

Figure 2.2 shows the numbers of impaired and unimpaired drivers involved in fatal wrong-way collisions on divided highways. Figure 2.2 shows that only a small percentage of the right-way drivers were impaired.

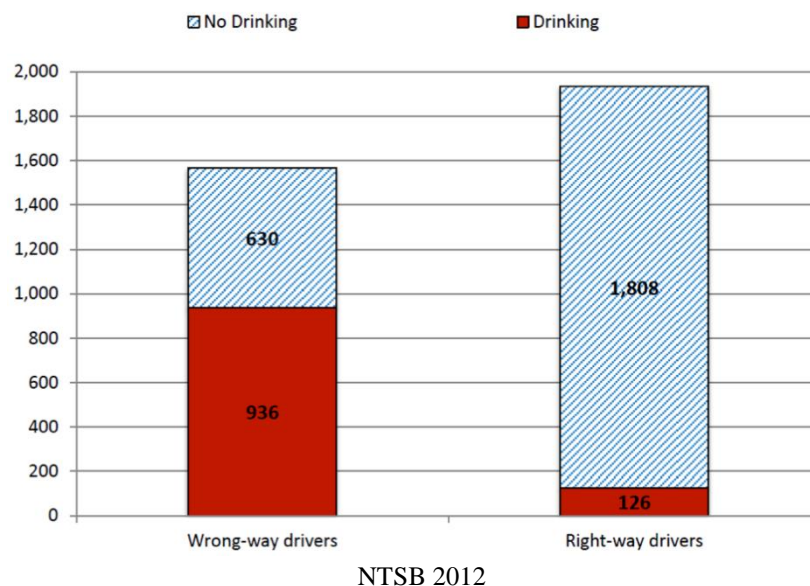


Figure 2.2. Wrong-way fatal crashes caused by impaired drivers

2.1.4 Presence of Passenger

About 85% of wrong-way crashes involved drivers with no passengers, indicating the possibility that passengers could aid in the prevention of wrong-way crashes (NTSB 2012).

2.1.5 Vehicle

A study using Illinois data found that passenger cars were the most common type of vehicle involved in wrong-way crashes. Table 2.1 shows the percentages of wrong-way crashes by vehicle type in Illinois (Zhou et al. 2012). The small number of commercial vehicles involved in wrong-way crashes may be explained by the fact that commercial drivers are highly regulated, but the difference between passenger vehicles and pickups/SUVs/minivans is more difficult to explain.

Table 2.1. Vehicle type for wrong-way crashes

Vehicle Type	Crash	
	Frequency	Percent
Passenger	139	68.5
Pickup	26	12.8
SUV	18	8.9
Van/minivan	12	5.9
Unknown	4	2.0
Tractor with semi-trailer	2	1.0
Motorcycle (over 150 cc)	1	0.5
Tractor without semi-trailer	1	0.5
Total	203	100

Source: Zhou et al. 2012

2.1.6 Facility

The type of roadway facility and location on the facility play an important role in wrong-way driving. Research conducted in California (Copelan 1989) and Texas (Conner et al. 2004) found that urban areas have significantly more wrong-way crashes than rural areas. NTSB (2012) reports the main findings of research on wrong-way crashes at interchange facilities as follows:

- Full, four-quadrant cloverleaf ramps have the lowest wrong-way entry rate, and left-hand exit ramps have the highest (NTSB 2012, Lew 1971)
- Partial interchanges have twice the wrong-way entry rate of full interchanges (NTSB 2012, Tamburri and Lowden 1968)
- High rates of wrong-way entry occur at incomplete interchanges and at loop exit ramps with crossroad terminals adjacent to the entrance ramp (NTSB 2012, Parsonson and Marks 1979)
- Exit ramps that terminate at two-way streets have high wrong-way entry rates (NTSB 2012, Lew 1971)
- Interchanges with short sight distances at their decision points have a disproportionately high number of wrong-way movements (NTSB 2012, Copelan 1989)

- Exit ramps with rounded corners tend to encourage rather than deter wrong-way movements (Vaswani 1973). Because rounded corners provide less of a distinction between the roadway and the ramp than sharp corners, they may mislead drivers into continuing along their current path of travel and thus mistakenly entering the exit ramp (NTSB 2012)

Zhou et al. (2015) investigated wrong-way entry points by interchange type in Illinois. Table 2.2 shows the interchange types and the corresponding entry points reported in Zhou et al. (2015).

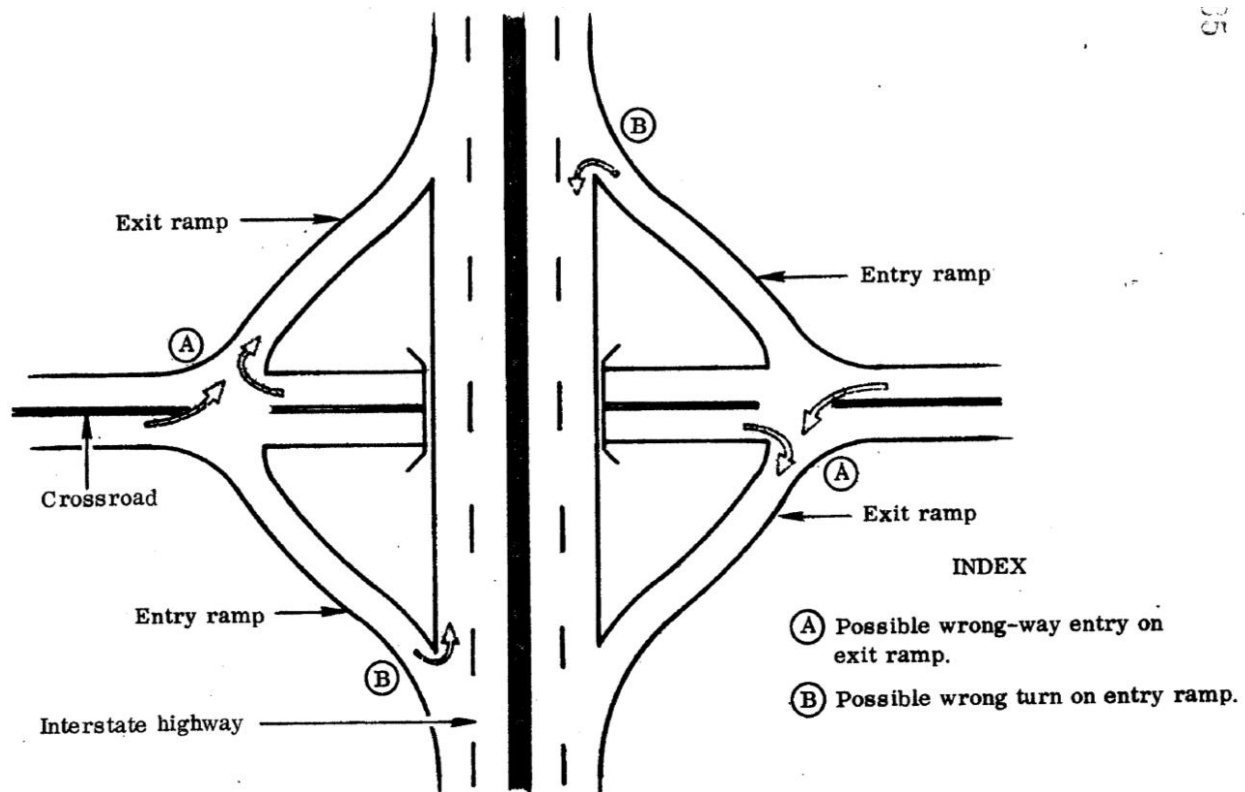
Table 2.2. Wrong-way crash entry points by interchange type

Interchange Type	Recorded		1st Estimated Entry Point		2nd Estimated Entry Point		Total No. of Interchanges in Illinois		WW Crash Rate	Rank
	#	%	#	%	#	%	#	%	% per year	
Compressed Diamond	12	25.53	44	29.93	44	30.14	56	7.64	13.39	1
Diamond	16	34.04	39	26.53	38	26.03	308	42.02	2.44	6
Partial Cloverleaf	5	10.64	28	19.05	23	15.75	79	10.78	5.22	3
Cloverleaf	3	6.38	12	8.16	12	8.22	59	8.05	3.39	5
Reset Area	1	2.13	9	6.12	6	4.11	64	8.73	1.82	6
Freeway Feeder	5	10.64	3	2.04	6	4.11	30	4.09	4.44	4
Modified Diamond	3	6.38	4	2.72	4	2.74	61	8.32	1.64	6
Semi-Directional	0	0.00	3	2.04	4	2.74	19	2.59	2.19	6
SPUI	1	2.13	2	1.36	3	2.05	8	1.09	5.73	2
Trumpet	0	0.00	2	1.36	4	2.74	25	3.41	1.33	7
Directional	1	2.13	1	0.68	2	1.37	24	3.27	1.39	7
Total	47	100.00	147	100.00	146	100.00	733	100.00	3.57	-

Source: Zhou et al. 2015

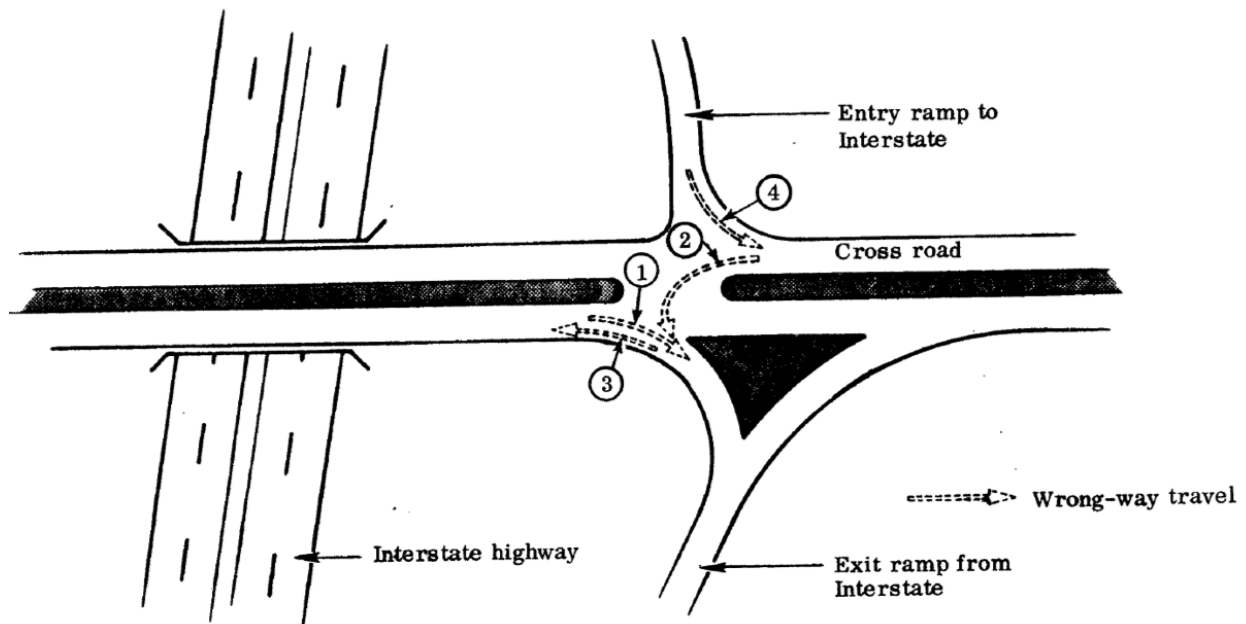
The right-most column in Table 2.2 lists the ranks of different designs based on wrong-way crash rate. The compressed diamond, single-point urban interchange (SPUI), and partial cloverleaf designs are the top three ranked interchange types, which indicates that wrong-way crashes are most frequent at these interchanges. Diamond interchanges, which are the most common type in Illinois with 308, have a lower crash rate than many other interchange types, including the full cloverleaf design.

Additionally, Vaswani (1973) studied the possible wrong-way entries at an Interstate highway interchange. Figure 2.3 shows the possible entry points at a conventional diamond interchange, and Figure 2.4 shows in more detail the wrong-way entry points for left and right turning vehicles at a ramp terminal.



Vaswani 1973, Virginia Highway Research Council

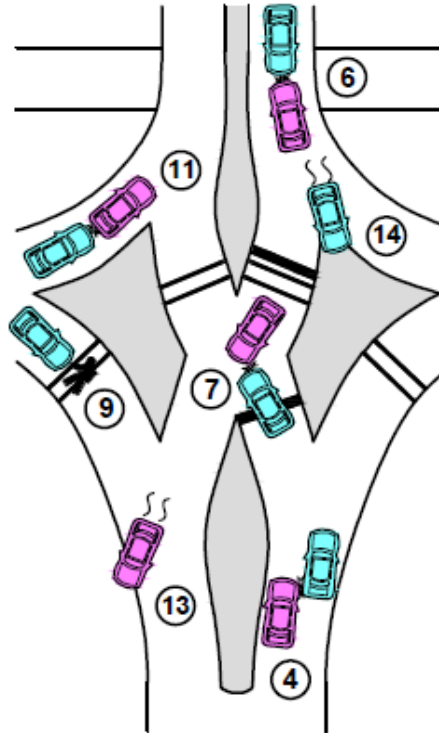
Figure 2.3. Wrong-way entry points at a diamond interchange



Vaswani 1973, Virginia Highway Research Council

Figure 2.4. Wrong-way entry points at a ramp terminal

With the increasing popularity of diverging diamond interchanges (DDI), there has been a growing concern about wrong-way crashes because of the type of geometric configuration at such ramp terminals. Recent research performed in Missouri found that wrong-way crashes constituted 4.8% of the fatal and injury crashes that occurred at this type of ramp terminal. These crashes are due to wrong-way driving on the crossroad between the ramp terminals when vehicles first enter the crossover intersection. Figure 2.5 illustrates the types of crashes occurring at DDI ramp terminals (Claros et al. 2015). The crash type labeled 6 shows the typical location of a wrong-way crash on the cross road.



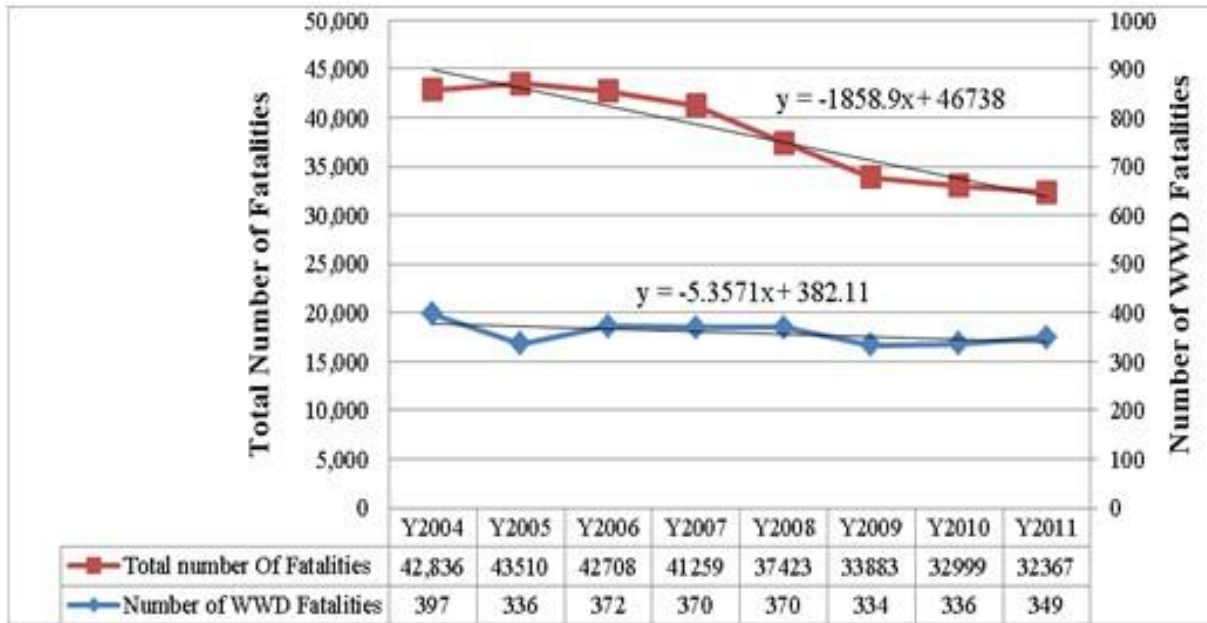
Claros et al. 2015, Transportation Research Board

Figure 2.5. DDI crash types

Vaughan et al. (2015) monitored traffic movements and conflicts at five DDIs in different states during a six-month period. Video recordings of ramp terminals were processed using video detection. The authors found a high number of wrong-way maneuvers at the five sites, but none of them resulted in a crash. The authors also reported that wrong-way maneuvers were more frequent during the night.

2.2 Wrong-Way Crash Statistics

Wrong-way crashes have a higher risk of resulting in severe injuries and fatalities. Using nationwide crash data from 2004 to 2011, Ghorghi et al. (2014) found that the total number of fatal crashes decreased, but fatal wrong-way crashes remained fairly constant during that same period (see Figure 2.6).



Ghorghi and Rouholamin 2013, Southern Illinois University - Edwardsville

Figure 2.6. Trends in total number of fatalities and number of wrong-way fatalities

While the reduction in the total number of fatal crashes can be attributed to the various safety countermeasures adopted by safety professionals, the lack of a decline in wrong-way driving fatal crashes shows a need to address this crash type.

Ghorghi et al. (2014) further reported that 57% of wrong-way driving fatal crashes occurred on urban roads and 43% occurred on rural roads, although only 24% of highway miles are designated as urban. A few studies have analyzed the time of occurrence of wrong-way driving crashes. Cooner et al. (2004) reported that 52% of crashes occurring between midnight and 6:00 a.m. in Texas were attributed to wrong-way driving; only 10.4% of all freeway crashes occurred during the same time period. In North Carolina, Braam (2006) reported that 33% of wrong-way crashes occurred during dark conditions without any street lighting and 28% occurred at night on roads with lighting.

Zhou et al. (2012) grouped the contributing factors of wrong-way crashes into six categories: (1) traffic violation, (2) inattention, (3) impaired judgment, (4) insufficient knowledge, (5) infrastructure deficiency, and (6) other factors, such as inclement weather. Traffic violation includes impaired driving and reckless driving. Inattention includes distracted driving and falling asleep at the wheel. Impaired judgment includes ill drivers and elderly drivers. Insufficient knowledge includes a lack of understanding of highway driving and a lack of familiarity with the facility. Infrastructure deficiency includes insufficient sight distance and lighting.

2.3 Countermeasures

Braam (2006) reported that wrong-way crashes are spread out over several miles of freeways with no identifiable concentrations, thus making the selection of treatment locations challenging.

A few states have implemented countermeasures to address wrong-way driving crashes and have reported their effectiveness. This section reviews countermeasures involving signage, geometric design, intelligent transportation systems (ITS) technology, and driver behavior.

2.3.1 Signing

The *Manual on Uniform Traffic Control Devices* (MUTCD) (FHWA 2012) provides guidance on signage and pavement markings to prevent wrong-way driving. There are two types of signage that are available to prevent wrong-way crashes at ramp terminals: minimum required and optional.

2.3.1.1 Minimum Required Signing

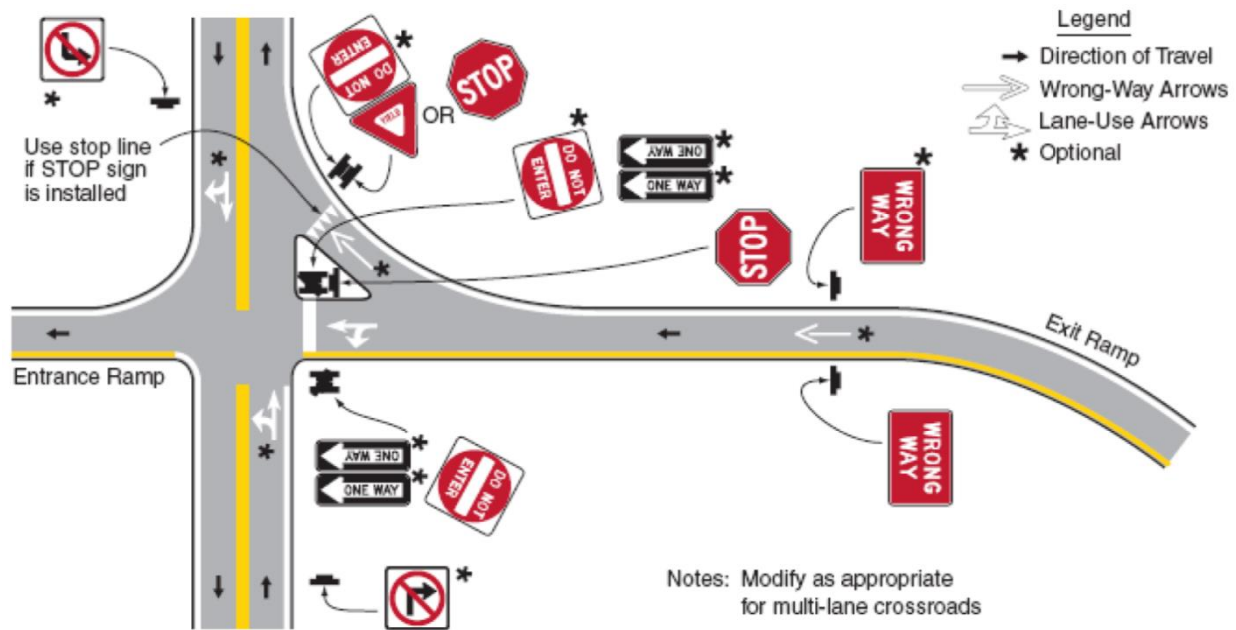
The minimum required signage for exit ramps that intersect with the crossroad should be installed, with a single One Way sign (R6-1), a single Do Not Enter sign (R5-1), and a single Wrong Way sign (R5a-1) (NTSB 2012, FHWA 2012), as shown in Figure 2.7.



Figure 2.7. Signing at exit ramps: R6-1 One Way (left), R5-1 Do Not Enter (center), R5a-1 Wrong Way (right)

2.3.1.2 Optional Signing

Optional signage additions include turn prohibition signs on the crossroad: No Right Turn or No Left Turn. Pavement markings include a slender and elongated wrong-way arrow or bidirectional red and white raised pavement markers in the shape of an arrow (NTSB 2012, FHWA 2012). Figure 2.8 shows both minimum and optional signage at a ramp terminal.



FHWA 2012 (from NTSB 2012)

Figure 2.8. MUTCD required and optional signing and paving marking at a ramp terminal

The NTSB (2012) recommends, as an option, doubling the number of minimum required signs at candidate locations, as shown in Figure 2.9.



New York State DOT (NYSDOT) (from NTSB 2012)

Figure 2.9. Optional double-posted Do Not Enter sign (R5-1) and Wrong Way sign (R5a-1)

The NTSB (2012) notes that many states have adopted innovative signage strategies for controlled-access highway interchanges to reduce wrong-way driving. The strategies are as follows:

- Lowering the height of Do Not Enter and Wrong Way signs. The minimum sign mounting height is 5 ft in rural areas and 7 ft when the line of sight is obstructed by parked vehicles or pedestrian movements. There is a provision in the MUTCD (Section 2B. 41) to lower the signs located along an exit ramp to 3 ft if an engineering study indicates that a lower mounting height would address wrong-way driving on freeway or expressway exit ramps.
- Using oversized Do Not Enter and Wrong Way signs (36 versus 30 in.) (FHWA 2012)
- Mounting both Do Not Enter and Wrong Way signs on the same post, paired on both sides of the exit travel lane i.e., the ramp
- Implementing a standard wrong-way sign package with larger dimension signs and twice the number of signs required by the MUTCD
- Implementing illuminated Wrong Way signs that flash when a wrong-way vehicle is detected
- Installing a second set of Wrong Way signs on the exit ramp farther upstream from the crossroad
- Posting controlled-access highway entrance signs on each side of an entrance ramp (FHWA 2012)
- Applying red retroreflective tape to the vertical posts of exit ramp signs
- Installing red delineators on each side of an exit ramp
- Installing LED-illuminated in-pavement markers or delineators parallel to the stop bar at the crossroad end of an exit ramp
- Installing trailblazing lines or reflective markers that channel travel in an arc to guide motorists making a left turn from the crossroad into an entrance ramp and thus to keep them from inadvertently entering an exit ramp (Morena and Leix 2012)

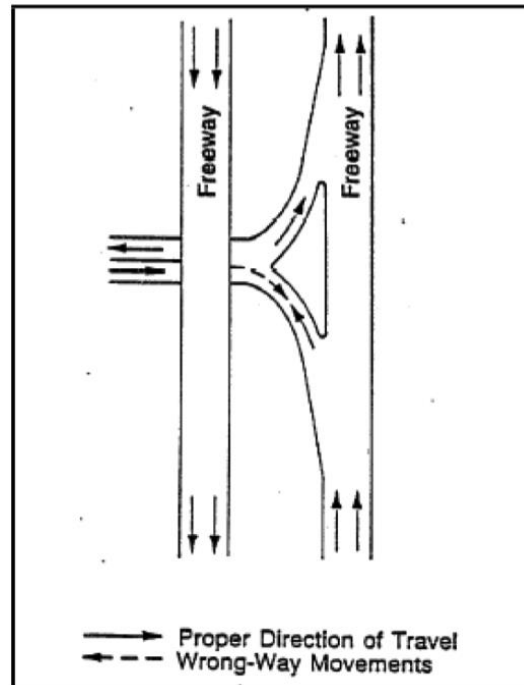
2.3.2 Geometric Countermeasures

The existing literature makes some recommendations on geometric design countermeasures that address wrong-way driving crashes. These countermeasures are as follows:

- Avoid left-side freeway exit ramps
- Install raised medians
- Use channelization devices
- Use tighter corner radii at exit ramp terminals
- Improve sight distance at intersections

2.3.2.1 Avoiding Left-Side Freeway Exit Ramps

Research performed in Texas and California (Cooner et al. 2004, Copelan 1989) found that left-side exit ramps on freeways can cause driver confusion and contribute to wrong-way driving. Figure 2.10 illustrates how the typical expectation of drivers to enter a freeway from the right-hand side can result in wrong-way entry at a left-side exit ramp.



Cooner et al. 2004, Texas Transportation Institute

Figure 2.10 Wrong-way movement at a left-hand freeway exit ramp

2.3.2.2 Raised Median

Cross streets at interchanges with traversable medians may result in wrong-way entries into exit ramps. This situation can be avoided by installing non-traversable medians on cross streets, thus making it physically challenging for vehicles to make a wrong-way maneuver. Figure 2.11 shows an example of a non-traversable median (Pour-Rouholamin and Zhou 2015), with a wrong-way maneuver shown in red and a safe maneuver shown in green.



Pour-Rouholamin and Zhou 2015

Figure 2.11. Raised median implemented at intersection

2.3.2.3 Channelization

Similar to raised medians, channelization devices can be used to discourage wrong-way turning movements. The use of longitudinal delineators for a left-turn lane can direct traffic into the desirable turning path (see Figure 2.12).



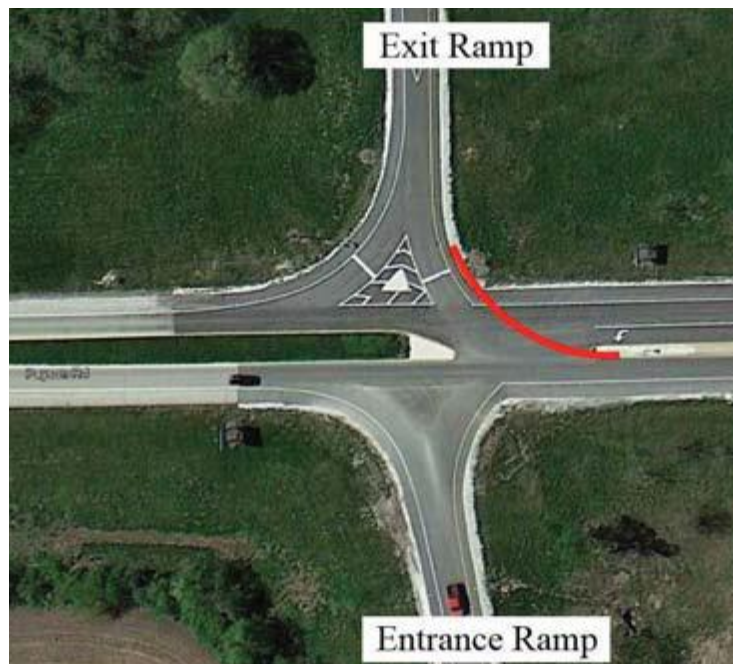
Pour-Rouholamin and Zhou 2015

Figure 2.12. Channelization at an intersection

Another common channelization treatment is the use of islands. A height of at least 4 in. is recommended because vehicles may drive over lower islands.

2.3.2.4 Tighter or Angular Corner Radii at Exit Ramp Terminals

The radius at the corner of intersecting roads can be used to prevent wrong-way movements. At ramp terminals, the corner radius can discourage right turning movements in the wrong direction from the crossroad to the exit ramp (NTSB 2012, Pour-Rouholamin and Zhou 2015). Guidance suggests that larger circular radii may encourage wrong-way movements; therefore, angular or tight radii make this movement difficult and have been found to be effective in states like Virginia for reducing wrong-way entry (Pour-Rouholamin and Zhou 2015, Vaswani 1977). An example of a sharper corner radius to discourage a wrong-way turn is shown in Figure 2.13.



Pour-Rouholamin and Zhou 2015

Figure 2.13. Radius treatment at ramp terminals

2.3.2.5 Sight Distance

Providing adequate sight distance at intersections allows drivers to identify the traffic control and geometric features of roadway facilities. Improving lighting, removing obstructions limiting sight distance, and placing stop bars and signal heads appropriately are all helpful measures to discourage wrong-way entry at intersections.

2.3.3 ITS Countermeasures

Many devices and technologies have been developed over the years to address wrong-way crashes. Some ITS countermeasures include in-vehicle alerts based on GPS, video-based detection and alerts, and in-pavement sensors and radar sensors to detect and alert drivers. Due to the high installation and maintenance costs of ITS devices, it may not be cost-effective to deploy

ITS countermeasures on a system-wide basis. A more feasible approach would be to deploy them at locations with a history of wrong-way driving crashes.

2.3.3.1 Wrong-Way GPS Vehicle Alerts

Several automobile companies have invested in developing wrong-way alert systems using GPS devices embedded in vehicles. Nissan, Toyota, and BMW have independently developed GPS-based alerts. Some of these technologies are already operational in countries like Japan and will soon be available in the United States (NTSB 2012). The NTSB has reported that wrong-way alerts using GPS systems on vehicles are effective and reliable (NTSB 2012).

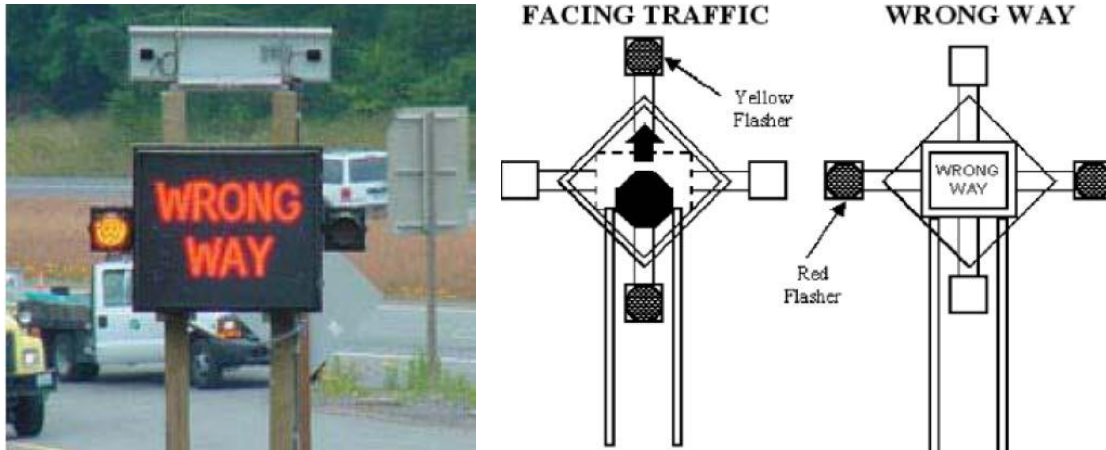
2.3.3.2 Video-Based Detection and Alerts

Video-based detection and alert systems rely on cameras deployed to monitor ramp vehicles. Image processing software is used in real-time to detect any vehicles going the wrong way. If a wrong-way driver is detected, alerts are sent to the local traffic management center (TMC) and police department and to nearby dynamic message signs. The wrong-way driver is also alerted using flashing lights installed on signs adjacent to the ramp. Some deployments have complemented such signs with Wrong Way LED signs.

The Washington State DOT (WSDOT) tested a video detection and warning system at the I-90/161st Avenue Southeast interchange in the Seattle, Washington, area. When a wrong-way movement was detected, a message sign was activated that flashed a Wrong Way message to the wrong-way driver (Zhou et al. 2012). Video-based detection systems have some limitations with respect to their need for ambient lighting during different times of day and weather conditions.

2.3.3.3 In-pavement Sensors and Alerts

WSDOT tested pavements embedded with electromagnetic sensors at the I-5/Bow Hill Road interchange near Edison, Washington, to detect wrong-way movements. A mounted dynamic sign with flashing lights was installed at the exit ramp to alert wrong-way drivers. Figure 2.14 (left) shows the dynamic sign. The state of New Mexico also tested a wrong-way alert system based on data from loop detectors and dynamic signs on both sides of an exit ramp, as shown in Figure 2.14 (right) (Zhou et al. 2012, Cooner et al. 2004).



Dawn McIntosh, WSDOT (from Moler 2002) (left) and Zhou et al. 2012, Illinois Center for Transportation (right)

Figure 2.14. Dynamic sign in Washington (left) and directional traffic sign in New Mexico (right)

2.3.3.4 Radar and Warning Alerts

Radar detection of wrong-way drivers has been tried in a few states, including Florida and Texas. Unlike video-based detection, the performance of radar systems is not sensitive to weather conditions or lighting.

The Florida DOT (FDOT) installed a radar-based wrong-way driving detection and warning system on Pensacola Bay Bridge in Pensacola, Florida. The system alerted drivers using signs and overhead flashing lights. For the wrong-way driver, the alerts include a combination of Do Not Enter and Wrong Way signs with flashing lights. Overhead flashing lights are used to alert traffic traveling in the correct direction of wrong-way vehicles. Figure 2.15 shows the system with overhead signs (Zhou et al. 2012, Cooner et al. 2004, Williams 2006).



Williams 2006, FDOT (from Zhou et al. 2012)

Figure 2.15. Wrong-way driver warning system at Pensacola Bay Bridge in Florida

In Houston, Texas, a wrong-way detection and alert system was deployed on the Harries County Tollway. The deployment consisted of 12 microwave radar detectors that detected wrong-way drivers and alerted the local TMC. The TMC personnel then manually verified the event using CCTV footage. After verification, dynamic message signs alerted vehicles traveling in the correct direction of the approaching wrong-way vehicle. The TMC also immediately notified the police (Zhou et al. 2012, Pour-Rouholamin and Zhou 2015, NTTA 2009).

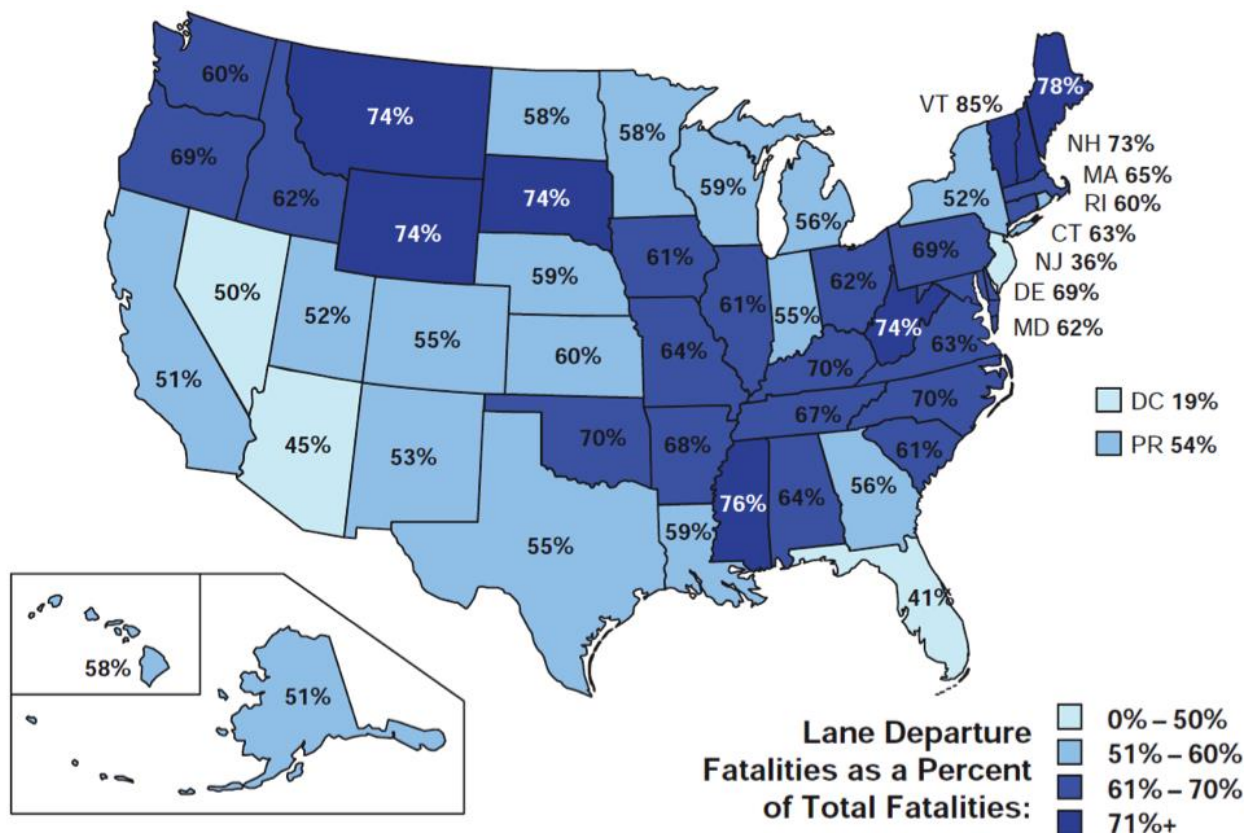
Additional ITS deployments are currently being planned in Texas (Zhou et al. 2012), Arizona (Simpson and Karimvand 2015), Florida (Sandt et al. 2015), and Germany (Oeser et al. 2015). However, the deployments have not yet been evaluated for their effectiveness.

2.3.4 Driver-Related Countermeasures

Even though driver-related countermeasures to combat wrong-way driving are not engineering countermeasures, a brief review of the main technologies is presented here to provide a better overall context of wrong-way driving countermeasures. Alcohol impairment is a major contributing factor to wrong-way crashes. Research using FARS data for 2004 to 2009 found that 9% of wrong-way drivers had been convicted of driving while intoxicated (DWI) within the three years prior to the wrong-way crash. This percentage was three times higher than that of a control group of drivers (NTSB 2012). NTSB has recommended the implementation of alcohol ignition interlock devices for several years. An alcohol ignition interlock is a device connected to the vehicle ignition circuit. It prevents the engine from starting unless a breath sample is determined to be lower than the state's blood alcohol limit. Alcohol ignition interlock devices have been developed for passenger vehicles (NTSB 2012, Jurnecka 2015, Blanco 2015) and for buses and commercial trucks (NTSB 2012, Podda 2012).

3. HORIZONTAL CURVES

Horizontal curves are of interest because these are frequently the site of road departure crashes that lead to severe injuries or fatalities. Around 4 out of every 10 fatal crashes involve vehicles leaving the roadway, and there are more than twice as many lane departure crashes on rural roads than on urban roads (AASHTO 2008). Some types of crashes involving lane departures are rollovers (42%) and collisions with trees (25%). In 2006, a total of 25,082 lane departure crashes were recorded, which represented 58% of total fatalities during that year (AASHTO 2008). Figure 3.1 shows the proportion of total traffic fatalities that are caused by lane departure crashes in each state.



From *Driving Down Lane-Departure Crashes: A National Priority*, Copyright 2008, by the American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.

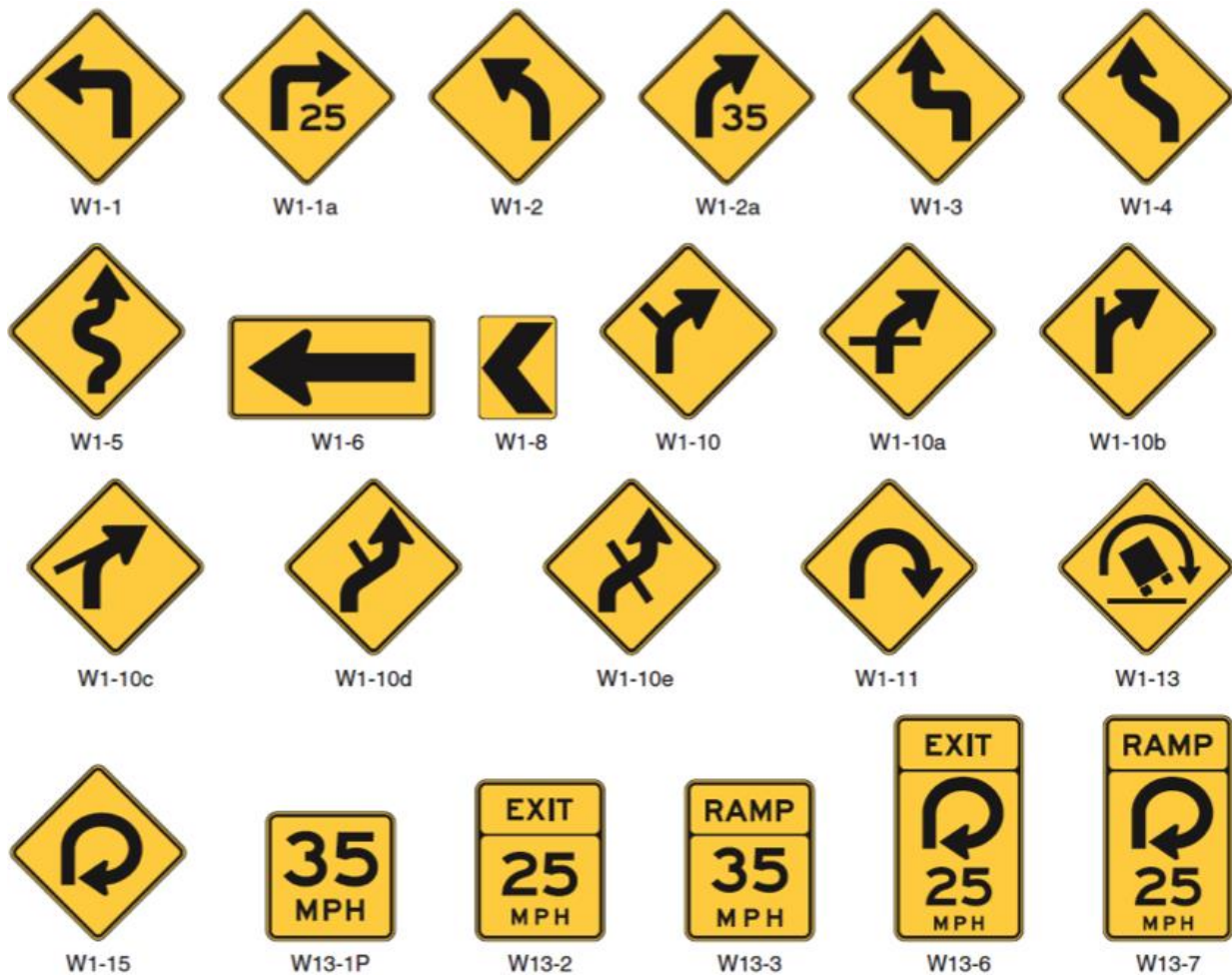
Figure 3.1. Lane departures fatalities in 2006

In Missouri, system-wide treatments such as cable median barriers and edge line rumble strips have been deployed on the primary roadway system. As a result, lane departure fatalities fell by 37% between 2005 and 2011 (MCRS 2012). The following discussion examines other system-wide treatments that can be applied in the state to further lower lane departure fatalities, especially on horizontal curves.

3.1 Signing

3.1.1 MUTCD Guidance

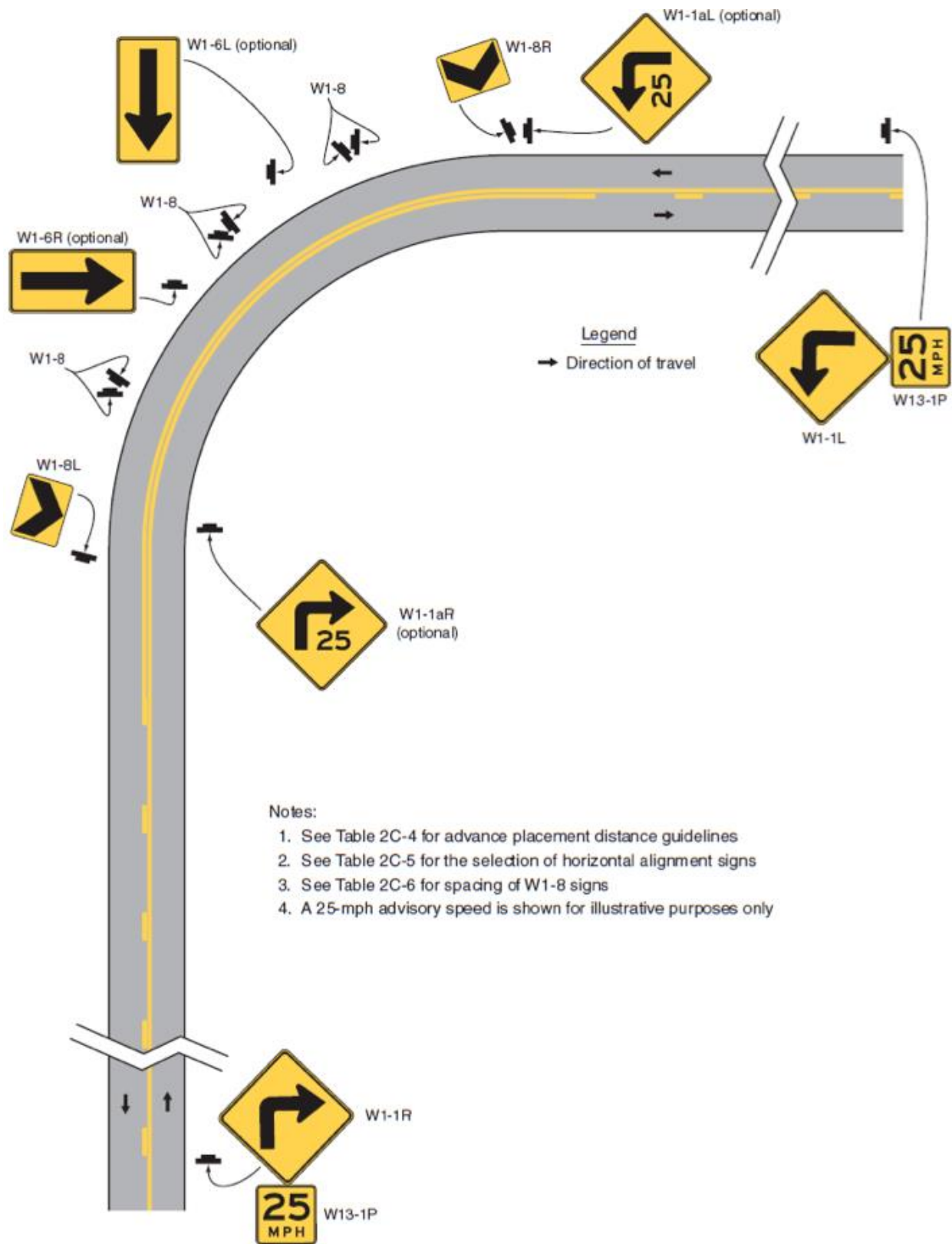
The MUTCD provides specific guidance for warning signs on horizontal curves. A combination of alignment warning signs, pavement markings, and delineation is recommended to provide guidance to drivers when driving through a horizontal curve (FHWA 2012). Figure 3.2 shows standard signs used on horizontal curves.



FHWA 2012

Figure 3.2. MUTCD horizontal curve warning signs

Selection of the applicable set of signs is based on the road's annual average daily traffic (AADT), functional classification, and posted or statutory speed limit or 85th percentile speed. If traffic is less than 1,000 AADT, the horizontal curve signing configuration is based on engineering judgment (FHWA 2012). Figure 3.3 provides an example of horizontal curve signing on a two-lane roadway.

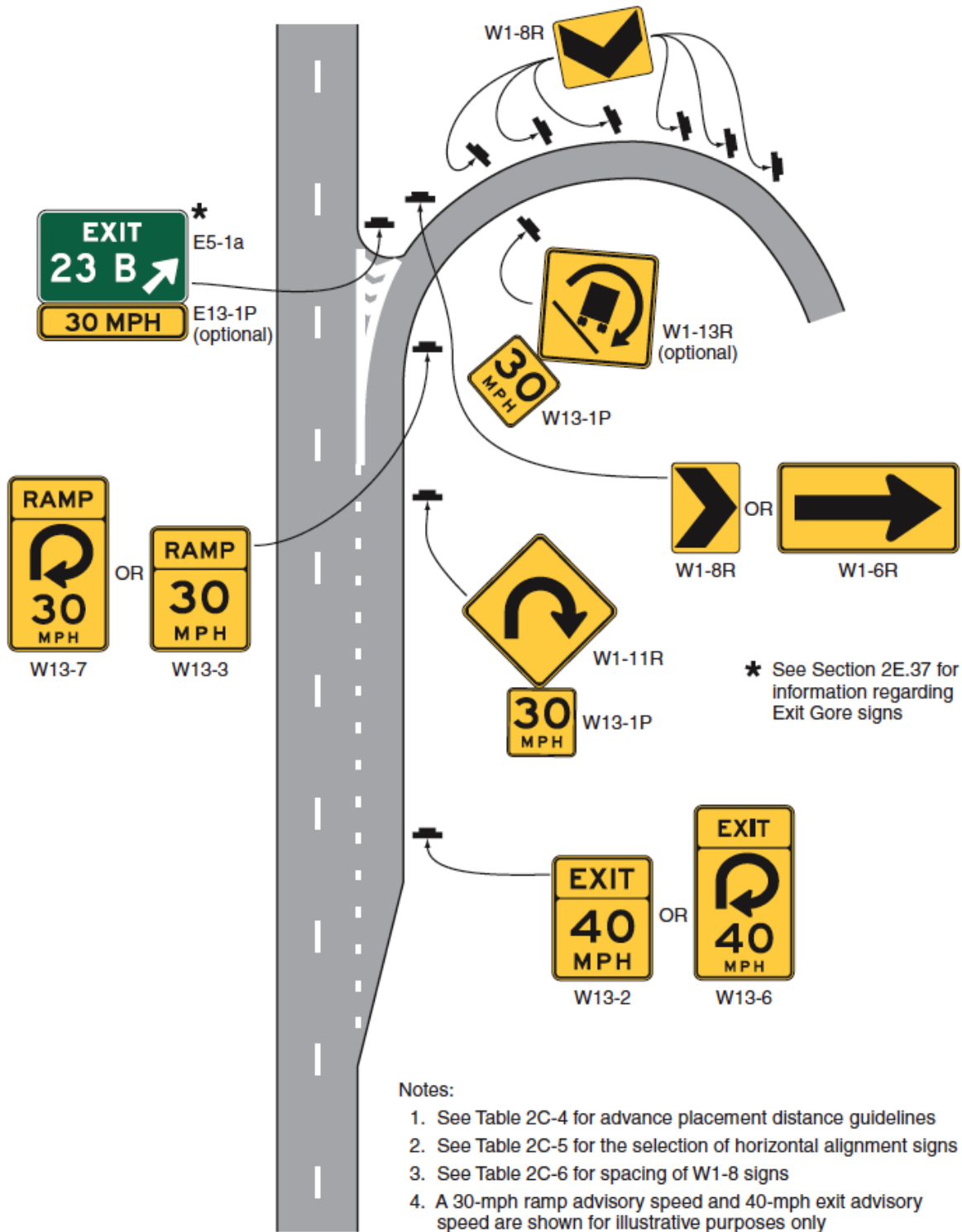


FHWA 2012

Figure 3.3. Example of MUTCD signing standards on a horizontal curve

On both approaches in Figure 3.3, a W1-1L/R combined with a W13-1P are provided upstream of the curve to warn drivers of the presence of the curve and the recommended speed. The curve may have another W1-1aR sign, as shown for the right turn, which reinforces the presence of the curve and the recommended speed at the beginning of the curve. Chevron signs (W1-8L/R) are provided along the curve, and directional signs (W1-6L/R) may be included to reinforce the direction of travel.

Another example of curve signage is shown in Figure 3.4 for exit ramp horizontal curves.



FHWA 2012

Figure 3.4. Example of MUTCD signing standards at an exit ramp

In Figure 3.4, warning signs are provided at the beginning of the taper for the speed-change lane and at the gore. The recommended speeds are based on the location of the facility. Chevron signs

are installed along the curve. Additional truck overturn warning signs with speed recommendations may be included.

Countermeasures targeting horizontal curve crashes may involve augmenting the minimum recommended MUTCD signs and devices at horizontal curves. Studies experimenting with and evaluating the safety effectiveness of such countermeasures were examined for this review.

3.1.2 Improved Curve Signing

In an FHWA pooled fund study of 26 states (Missouri was not a participant), low-cost safety treatments for improving curve delineation were examined by Srinivasan et al. (2009). Treatments on two-lane roads included the addition of new signs: chevrons, arrows, and advance warning. In addition, existing signs were made more retroreflective using fluorescent yellow sheeting. Data from deployments in Connecticut and Washington State were used to conduct a safety evaluation. An 18% reduction in injury and fatal crashes and a 25% reduction in lane departure crashes during dark conditions were achieved by improving curve delineation. Figure 3.5 provides an example of chevrons installed on a curve in Connecticut.



Srinivasan et al. 2009, FHWA Turner-Fairbank Highway Research Center

Figure 3.5. Example of improved curve signing in Connecticut

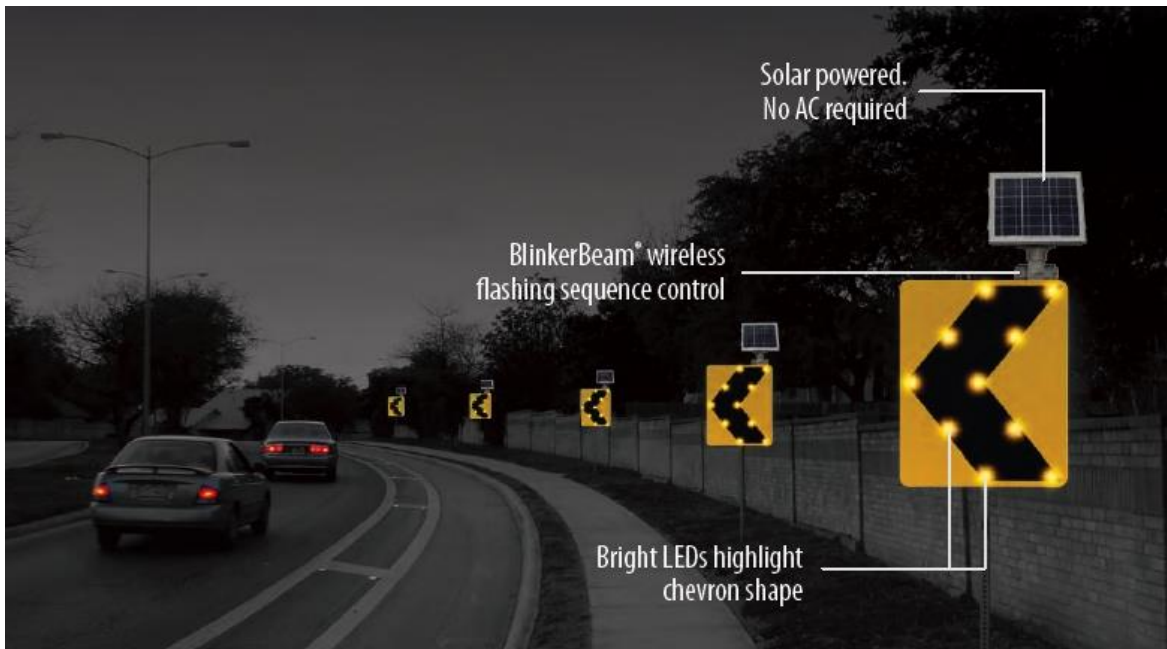
A study performed in Italy found that the installation of warnings signs, chevron signs, and sequential flashing beacons along horizontal curves reduced total crashes by 47.6%, injury crashes by 38.2%, and nighttime crashes by 76.9% (Montella 2009). In a Florida study, flashing beacons deployed on curves reduced the total number of crashes by 30% (Gan et al. 2005). Flashing beacons are signals that operate in a continuous flashing mode to warn drivers of the curve and the posted lower advisory speed limit (FHWA 2012). Figure 3.6 shows flashing beacons installed on both sides of a sharp curve.



Bowman n.d.

Figure 3.6. Flashing beacons

Dynamic flashing chevrons (see Figure 3.7) have been deployed on a few curves in Iowa, Missouri, Texas, Washington, and Wisconsin (Smadi et al. 2015).



Copyright 2012, Traffic and Parking Control Co, Incorporated

Figure 3.7. Dynamic curve guidance systems

These LED illuminated chevrons shown in Figure 3.7 are wirelessly synchronized and show drivers the direction of the curve. The treated sites witnessed a slight reduction in vehicle speeds, about 1 mph. Nine treatment sites experienced reductions in crashes ranging from 17% to 91%.

Two sites experienced an increase in crashes of 7% and 11%. Two other sites did not experience any crashes after the treatment.

Oversized chevrons are also good candidates for improving curve safety. The typical size of chevron signs (W1-8) specified in the MUTCD is 12 x 18 in., and oversized chevrons are 18 x 24 in. The larger signs provide greater sight distance to drivers. The MUTCD recommends that oversized signing be used when engineering judgment indicates the need for larger signing because of vehicle speeds, driver expectancy, traffic operations, or roadway conditions (FHWA 2012).

Dynamic speed warning systems for horizontal curves, as shown in Figure 3.8, have been piloted in Iowa.



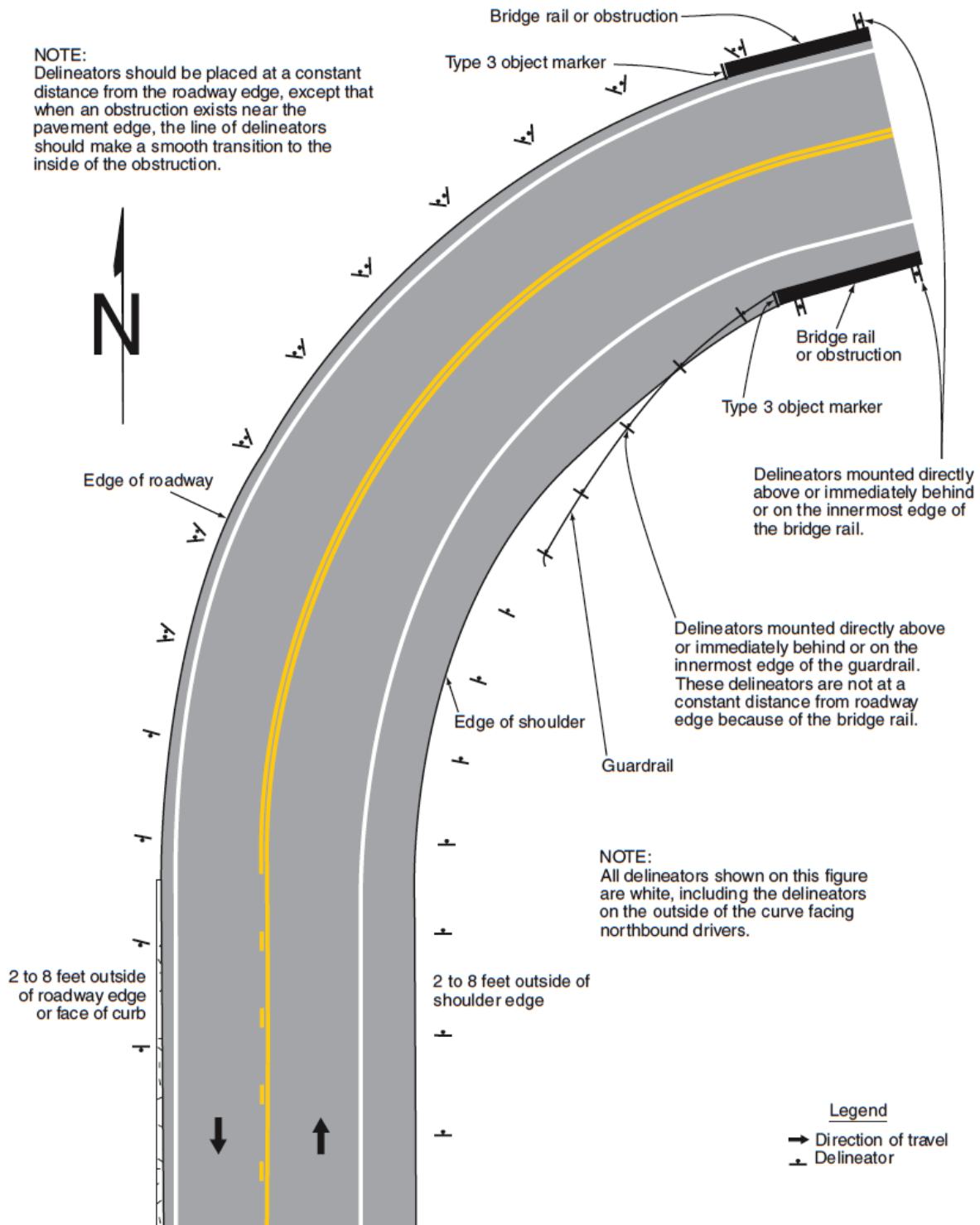
Hallmark et al., Institute for Transportation

Figure 3.8. Dynamic speed warning signs

These systems detect the speed of an approaching vehicle, display it on a LED panel, and can display a Slow Down LED sign, as shown in Figure 3.8. Hallmark et al. (2015) found that dynamic speed warning systems reduced total crashes by 5% to 7%. Moreover, these systems were found to reduce the proportion of drivers exceeding the posted speed limit (Hallmark et al. 2012a).

3.1.3 Vertical Delineation

Roadway delineation is used at locations where the alignment might be confusing or unexpected. Delineators are effective guidance devices at night and during adverse weather conditions. According to the MUTCD, retroreflective elements for delineators shall have a minimum dimension of 3 in. (FHWA 2012). Figure 3.9 provides an example of delineator placement on a curve (FHWA 2012).



FHWA 2012

Figure 3.9. Example of curve delineator deployment

While the use of delineators has not been shown to reduce crashes on curves, their use in combination with edge lines and centerlines reduced all fatal and injury crashes by 45% (CMF Clearinghouse n.d., Elvik et al. 2004).

Finally, the addition of retroreflective devices to chevron vertical posts has been found to slow down drivers negotiating curves. While studies have found this treatment to reduce vehicle speeds, it has not been found to reduce crash numbers (Hallmark et al. 2012b, Re et al. 2010, Vest et al. 2005). An example treatment in Iowa is shown in Figure 3.10.



Hallmark et al., Institute for Transportation

Figure 3.10. Chevron signs with retroreflective posts

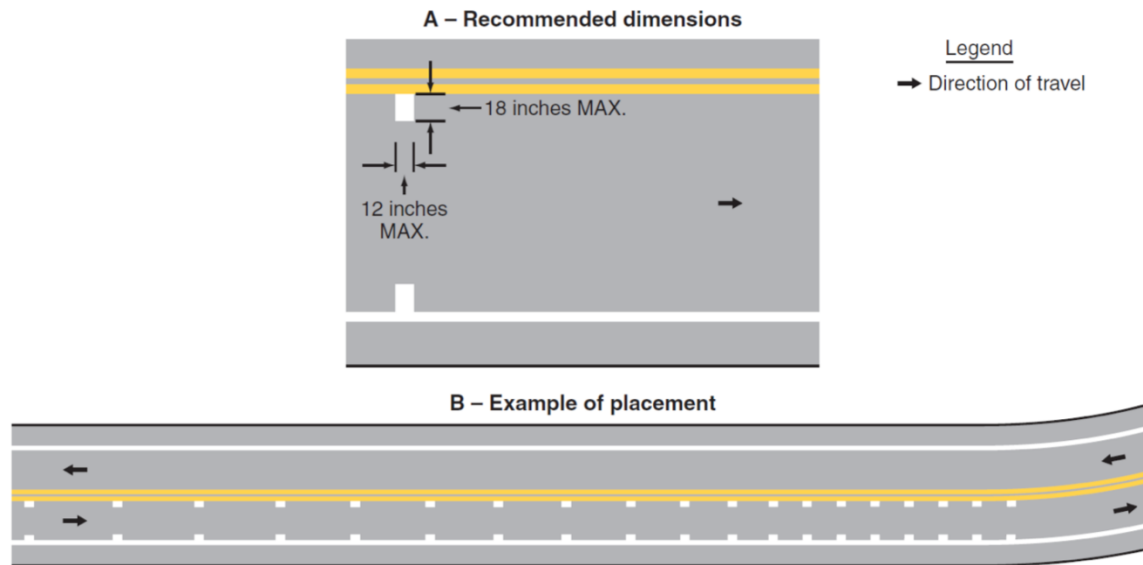
3.2 Pavement Markings and Treatments

3.2.1 Wide Edge Lines

The width of pavement line markings indicates the marking's degree of emphasis. Edge line pavement markings delineate the right and left edges of a roadway. Edge lines provide a visual reference to guide users during adverse weather and reduced visibility conditions. Wider edge line markings may be used for greater emphasis of the roadway's edges. The MUTCD requires a width of 4 to 6 in. for normal edge lines and double that size (i.e., 8 to 12 in.) for wide edge lines (FHWA 2012). Widening of edge lines has been found to (1) slow down drivers earlier when they enter a horizontal curve (McGee and Hanscome 2006), (2) decrease crashes with fixed objects by 17% (Donnell et al. 2006), and (3) decrease nighttime crashes (Tsyganov et al. 2005).

3.2.2 Speed Reduction Markings

Speed reduction markings are a pavement marking treatment used to slow down drivers approaching a sharp horizontal curve. As shown in Figure 3.11, these transverse markings are placed along both edges of the lane, with the spacing decreasing as drivers negotiate the curve (FHWA 2012).



FHWA 2012

Figure 3.11. Application of speed reduction markings

The MUTCD provides the following guidance: “If used, speed reduction markings shall be a series of white transverse lines on both sides of the lane that are perpendicular to the center line, edge line, or lane line. The longitudinal spacing between the markings shall be progressively reduced from the upstream to the downstream end of the marked portion of the lane” (FHWA 2012).

A reduction in 85th percentile speed of up to 5 mph has been reported with the use of speed reduction markings (FHWA 2012, Tsyganov et al. 2005, Hallmark et al. 2007). One study reported a 57% reduction in speed-related crashes due to the deployment of speed reduction markings on roundabout approaches (Griffin and Reinhardt 1996). A variation of the MUTCD speed reduction marking, where the transverse markings extend across most of the lane width (see Figure 3.12), has also been found to reduce vehicle speeds on horizontal curves (Vest et al. 2005, Katz et al. 2006, Arnold and Lantz 2007).

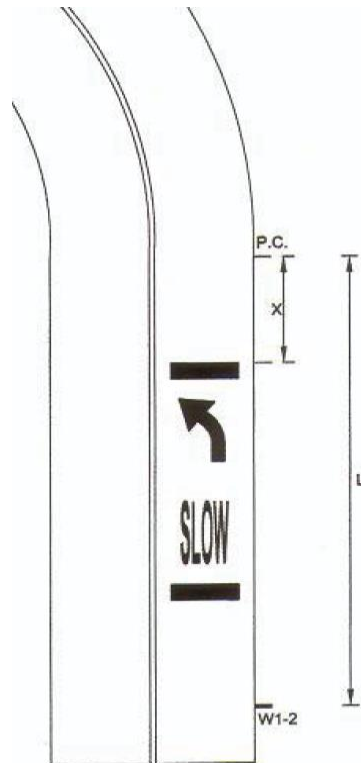


Arnold and Lantz 2007

Figure 3.12. Variation of speed reduction markings

3.2.3 Words and Symbols

Some states have tried deploying combinations of MUTCD pavement marking symbols and words on horizontal curves. Figure 3.13 illustrates the implementation of an experimental combination of pavement markings—the word SLOW with a curving arrow symbol—in Pennsylvania. A speed reduction of up to 10% from advanced symbol/word combinations was reported in several studies (McGee and Hanscom 2006, Chrysler and Schrock 2005, Retting and Farmer 1998, Nambisan and Hallmark 2011).



McGee and Hanscom 2006, FHWA

Figure 3.13. Example of a pavement marking warning symbol in combination with the word SLOW

3.2.4 Raised Pavement Markers

Retroreflective raised pavement markers (RPMs) are used to delineate the transition of a curve at night. They can be used along the roadway's centerline or edge line. A snowplow-durable RPM was recently studied for centerline deployment in rural areas by Bahar et al. (2004). For curves with a radius greater than 1,640 ft, the authors found a change in nighttime crashes between 33% and -13%, with the negative value indicating an increase in crashes. For curves with a radius smaller than 1,640 ft, nighttime crashes were found to change (increase) between -3% and -26%.

3.2.5 Rumble Strips and Stripes

Rumble strips and stripes are spaced transverse dents in the pavement that provide audible and tactile (vibration) alerts when vehicle tires roll over them. They have successfully been implemented in several states to prevent lane departures. Torbic et al. (2009) reported that the safety effectiveness of centerline rumble strips on horizontal curves (a 47% reduction in total target crashes) was similar to their effectiveness on tangent sections (a 49% reduction). A study in Minnesota evaluated crash rates before and after implementation of edge line rumble strips on curves and found a reduction in total crashes of 15% (Pitale et al. 2009).

3.2.6 High-Friction Pavement Treatments

High-friction pavement treatments work by increasing the pavement's friction, thus helping vehicles stay within the lane while negotiating a horizontal curve. Such treatments can be helpful during wet pavement conditions, when the friction between a vehicle's tires and the pavement is less than during dry conditions. Treatments are usually composed of a combination of resins, polymers with a binder, and aggregate.

Studies of treatments on freeway ramp curves have shown that high-friction pavement treatments have reduced total crashes by 25%, fatal and injury crashes on wet pavement by 14%, and fatal crashes on sharp curves by 25% (Nambisan and Hallmark 2011, Julian and Moler 2008). In New York, high-friction treatments were applied as part of a skid accident reduction program (SKARP). The application resulted in a 24% reduction in total crashes and a 57% reduction in crashes occurring in wet road conditions (Harkey et al. 2008).

4. INTERSECTIONS

This study examined treatments at both signalized and stop-controlled intersections. The following six signalized intersection treatments were reviewed: increasing clearance interval, changing left turn from permissive to permissive-protected, installing flashing yellow arrows, installing dynamic signal warning, installing red light cameras (RLCs), and improving signal visibility. The stop-controlled intersection treatments included stop sign improvements and flashing beacons.

4.1 Signalized Intersections

4.1.1 Increasing the Clearance Interval

The MUTCD states that the duration of yellow and red clearance intervals should be determined based on engineering practice. The yellow interval should be between 3 and 6 seconds, and intervals longer than 6 seconds must only be considered for approaches with higher speeds. Red clearance intervals should not exceed 6 seconds unless clearing one lane, two-way facilities, or wide intersections (FHWA 2012).

NCHRP 17-35 (Srinivasan et al. 2011) studied the effects of increasing yellow and red clearance times on intersection safety. A summary of the results is shown in Table 4.1.

Table 4.1. Safety effects of changing yellow and red clearance times

Treatment, Crash Type, and Severity	No. of Treated Sites	CMF (S.E. of CMF)
Increase Yellow and All Red (All)	11	0.991 (0.146)
Increase Yellow and All Red (Injury and Fatal)		1.020 (0.156)
Increase Yellow and All Red (Rear end)		1.117 (0.288)
Increase Yellow and All Red (Angle)		0.961 (0.217)
Increase Yellow Only (All)	5	1.141 (0.177)
Increase Yellow Only (Injury and Fatal)		1.073 (0.216)
Increase Yellow Only (Rear end)		0.934 (0.237)
Increase Yellow Only (Angle)		1.076 (0.297)
Increase All Red Only (All)	14	0.798 (0.074)*
Increase All Red Only (Injury and Fatal)		0.863 (0.114)
Increase All Red Only (Rear end)		0.804 (0.135)
Increase All Red Only (Angle)		0.966 (0.164)
Increase Change Interval (< ITE) (All)	12	0.728 (0.077)*
Increase Change Interval (< ITE) (Injury and Fatal)		0.662 (0.099)*
Increase Change Interval (< ITE) (Rear end)		0.848 (0.142)
Increase Change Interval (< ITE) (Angle)		0.840 (0.195)
Increase Change Interval (> ITE) (All)	15	0.922 (0.089)
Increase Change Interval (> ITE) (Injury and Fatal)		0.937 (0.114)
Increase Change Interval (> ITE) (Rear end)		0.643 (0.130)*
Increase Change Interval (> ITE) (Angle)		1.068 (0.156)

* Statistically significant at the 0.05 level

- The sample included 2 sites from Howard County, Maryland, 6 sites from Montgomery County, Maryland, 16 sites from San Diego, California, and 7 sites from San Francisco, California.
- In the before period, the average major road AADT was 17,417 (minimum major road AADT was 5,950 and maximum major road AADT was 31,600) and the average minor road AADT was 8,484 (minimum minor road AADT was 2,650 and the maximum minor road AADT was 20,225).
- Modifications to the yellow and all red time were not equivalent for all sites. For sites where both the yellow and all red time were increased, the average increases in the yellow and all red times were 0.8 seconds and 1.0 seconds, respectively. For sites where only the yellow interval was increased, the average increase in the yellow interval was 1.0 seconds. For sites where only the all red interval was increased, the average increase in the all red time was 1.1 seconds. For sites where the total change interval was increased, but still less than the ITE recommended practice, the average increase was 0.9 seconds. For sites where the total change interval was increased and exceeded the ITE recommended practice, the average increase was 1.6 seconds.
- The sample of sites used in this evaluation is limited. So these results should be used with due caution.

Methodology: Before-After Empirical Bayes

Source: Srinivasan et al. 2011

As Table 4.1 shows, when both yellow and red clearance intervals were increased, yellow by 0.8 seconds and red clearance by 1.0 seconds on average, there were modest reductions in angle and overall crashes and an increase in fatal and injury crashes and rear-end crashes. When only the

yellow interval was increased, on average by 1 second, there was an increase in overall crashes and fatal and injury crashes and a decrease in rear-end crashes. When only the red clearance interval was increased, on average by 1.1 seconds, all types and severities of crashes decreased (Srinivasan et al. 2011). The authors note that the small sample sizes used in the study contributed to the lack of statistical significance of most findings.

4.1.2 Changing Left Turn Phasing from Permissive to Protected-Permissive

NCHRP 17-35 (Srinivasan et al. 2011) studied changes in crashes due to the conversion of left turn phasing from permissive to protective-permissive at a few locations in Toronto and North Carolina. Table 4.2 provides a summary of the main findings (Srinivasan et al. 2011). The treatment was found to be successful at reducing the number of fatal and injury crashes. There were slight increases in the number of rear-end crashes and in the total number of crashes.

Table 4.2. Safety effects of changing left turn phase from permissive to protected-permissive

Number of Treated Approaches and Crash Type at Intersection Level	No. of Sites	CMF (S.E. of CMF)
All sites (all crashes)	71	1.031 (0.022)
1 treated approach (all crashes)	50	1.081 (0.027)*
>1 treated approach (all crashes)	21	0.958 (0.036)
All sites (injury and fatal crashes)	71	0.962 (0.035)
1 treated approach (injury and fatal crashes)	50	0.995 (0.043)
>1 treated approach (injury and fatal crashes)	21	0.914 (0.055)
All sites (left-turn opposing through crashes)	71	0.862 (0.050)*
1 treated approach (left-turn opposing through crashes)	50	0.925 (0.067)
>1 treated approach (left-turn opposing through crashes)	21	0.787 (0.072)*
All sites (rear-end crashes)	71	1.075 (0.036)*
1 treated approach (rear-end crashes)	50	1.094 (0.045)*
>1 treated approach (rear-end crashes)	21	1.050 (0.059)

* Statistically significant at the 0.05 level

- 59 intersections from Toronto and 12 from North Carolina. All of them were four leg intersections from urban areas.
- In Toronto, in the before period, the average major road AADT was 35,267 (minimum was 14,489 and maximum was 74,990) and the average minor road AADT was 18,096 (minimum was 1,466 and maximum was 42,723).
- In North Carolina, in the before period, the average major road AADT was 12,302 (minimum was 4,857 and maximum was 18,766) and the average minor road AADT was 5,124 (minimum was 1,715 and maximum was 9,300).
- It is important to note that left-turn phasing was not constant throughout the day for most of the sites (especially in Toronto), and hence, the sites were categorized based on the predominant phasing system.
- Among the 21 sites where more than 1 approach was treated, 17 of them had 2 approaches treated, 2 of them had 3 approaches treated, and 2 of them had 4 approaches treated.

Methodology: Before-After Empirical Bayes

Source: Srinivasan et al. 2011

4.1.3 Installing Flashing Yellow Arrows for Permissive Left Turns

A flashing yellow arrow at left-turn locations is designed to advise drivers of a permissive left turn and thus alert them to yield to oncoming traffic. NCHRP 17-35 used data from 55 treated sites in Washington, Oregon, and North Carolina. In locations where the signal configuration before the treatment was permissive or a combination of permissive and protective-permissive, total crashes and left turn crashes were reduced. In locations where the signal configuration was protected only, the installation of the flashing yellow arrow was found to increase total crashes, including left turn crashes. See Table 4.3.

Table 4.3. Safety effects of installing flashing yellow arrow

Left-Turn Phasing Before (sites) (legs treated)	Crash Type	CMF (S.E.)
Permissive or combination of permissive and protected-permissive (at least 1 converted leg was permissive in the before period) (9 sites) (20 legs treated)	Total Intersection Crashes	0.753 (0.094) *
	Total Intersection Left-Turn Crashes	0.635 (0.126) *
Protected-Permissive (all converted legs had protected-permissive in the before period) (13 sites) (27 legs treated)	Total Intersection Crashes	0.922 (0.104)
	Total Intersection Left-Turn Crashes	0.806 (0.146)
Protected (all converted legs had protected in the before period) (29 sites) (56 legs treated)	Total Intersection Crashes	1.338 (0.097) *
	Total Intersection Left-Turn Crashes	2.242 (0.276) *

* Statistically significant at the 0.05 level
Source: Srinivasan et al. 2011

4.1.4 Installing Dynamic Signal Warning Flashers

Dynamic signal warning flashers, which warn drivers that an upcoming traffic signal is turning red, are currently used in some states to enhance intersection safety. An example of this treatment is shown in Figure 4.1.



Srinivasan et al. 2011, National Cooperative Highway Research Program

Figure 4.1 Dynamic signal warning flasher

NCHRP 17-35 (Srinivasan et al. 2011) evaluated dynamic signal warning flashers implemented at sites in Nevada, Virginia, and North Carolina. The safety effectiveness results are shown in Table 4.4.

Table 4.4. Installation of dynamic signal warning flashers

	Total Crashes	Rear-end	Angle	Injury and Fatal	Heavy Vehicle
CMF	0.814*	0.792*	0.745*	0.820*	0.956
Standard Error	0.062	0.079	0.086	0.083	0.177

* Statistically significant at the 0.05 level (based on the ideal standard errors reported in this table)

Source: Srinivasan et al. 2011

A reduction in crashes was observed for all crash categories: total, rear-end, angle, injury and fatal, and heavy vehicle. Another study using Nebraska data (Appiah et al. 2011) also reported reductions in crashes due to this treatment.

4.1.5 Installing Red Light Cameras

RLCs are a treatment aimed at preventing drivers from running red lights, thereby preventing a severe angle crash. A comprehensive study of RLCs was conducted by Council et al. (2005), who analyzed data from 132 treatment sites and found that RLCs were successful at decreasing angle crashes. Rear-end crashes, however, increased after RLC installation. The study results are presented in Table 4.5.

Table 4.5. Aggregated red light camera safety effectiveness

	Right-angle		Rear end	
	Total crashes	(Definite) injury	Total crashes	(Definite) injury
EB estimate of crashes expected in the after-period without RLC	1,542	351	2,521	131
Count of crashes observed in the after-period	1,163	296	2,896	163
Estimate of percentage change (standard error)	- 24.6 (2.9)	- 15.7 (5.9)	14.9 (3.0)	24.0 (11.6)
Estimate of the change in crash frequency	- 379	- 55	375	32

A negative sign indicates a decrease in crashes

Source: Council et al. 2005

4.1.6 Improving Signal Visibility

Treatments that improve signal visibility include increasing signal lens size, adding backplates, adding reflective tape to existing backboards, and using an alternative signal configuration. Larger signal heads can be used to increase visibility and light output to provide awareness to drivers at greater distances (Sayed et al. 2007, Janoff 1994). The MUTCD contains standards regarding the location of signals. Table 4.6, from the MUTCD, shows the minimum sight distance necessary for a signal for a given 85th percentile speed. If the minimum sight distance is not met, a sign should be installed to warn drivers of the traffic signal (FHWA 2012).

Table 4.6. Minimum sight distance for signal visibility

85th Percentile Speed	Minimum Sight Distance
20 mph	175 feet
25 mph	215 feet
30 mph	270 feet
35 mph	325 feet
40 mph	390 feet
45 mph	460 feet
50 mph	540 feet
55 mph	625 feet
60 mph	715 feet

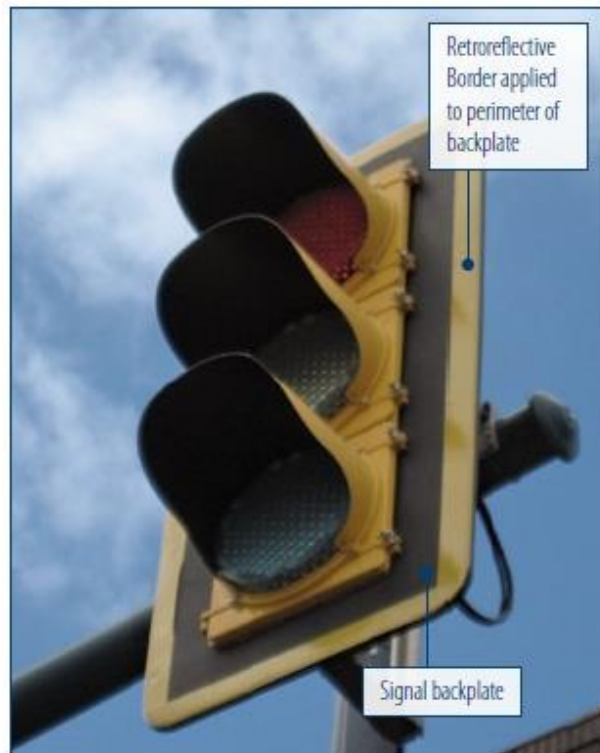
Distances derived from stopping sight distance plus an assumed queue length for shorter cycle lengths (60 to 75 seconds)

Source: FHWA 2012

A study concerning improved signal visibility considered 171 intersections (8 municipalities) in Canada. The researchers improved or increased signal lens size and added reflective tape to existing or new backboards. The results of the study showed an 8.5% reduction in property

damage only crashes, a 5.9% reduction in daytime crashes, a 6.6% reduction in nighttime crashes, and a 7.3% reduction in overall crashes (Sayed et al. 2007).

Signal heads with backplates and retroreflective edges have also been encouraged by the FHWA because they improve signal visibility and conspicuity for older and colorblind drivers. The addition of a reflective edge is even more advantageous during power outages when the signals are not operational (FHWA 2014a). The MUTCD recommends augmenting backplates with a one- to three-inch-wide yellow retroreflective edge border, as shown in Figure 4.2.



FHWA 2014a

Figure 4.2. Signal head with backplate and retroreflective edges

Sayed et al. (2005) reported a 15% reduction in crashes due to backplate treatments, and an FHWA (2010b) study reported even higher reductions of 28.6% for total crashes, 36.7% for injury crashes, and 49.6% for nighttime crashes.

4.2 Stop-Controlled Intersections

4.2.1 Improvement of Stop Signs

Stop signs can be enhanced for better visibility by increasing their size and retroreflectivity and by installing LED lights. While there are no formal studies quantifying the benefits of enhancing the size of stop signs, larger signs have been used in many states to increase their visibility

(Amparano and Morena 2006). Persaud et al. (2007) conducted a safety evaluation of increasing the retroreflectivity of stop signs. The data set for the evaluation consisted of 231 sites in Connecticut and 108 sites in South Carolina. The results of the study showed a statistically significant reduction in rear-end crashes in South Carolina. Three-leg and low-volume configurations especially experienced reductions in crashes. A slight reduction in nighttime crashes was also recorded in both states. The use of flashing LED lights on stop signs, as shown in Figure 4.3, has been found to reduce the failure to stop (Gates et al. 2003).



Arnold and Lantz 2007

Figure 4.3. Stop sign with LED lights

A study performed in Minnesota that included 15 intersections reported an estimated reduction in right-angle crashes of 41.5% due to the installation of LEDs on stop signs (Davis et al. 2014). No significant changes in speed, deceleration, and compliance were observed in the Minnesota study.

4.2.2 Flashing Beacons

The use of flashing beacons at stop-controlled intersections can bring heightened driver awareness to the presence of the intersection. Srinivasan et al. (2008) conducted a safety evaluation using three types of flashing beacons: overhead signals, signals on top of stop signs, and actuated flashers with a Vehicles Entering When Flashing sign. Flashing beacons were deployed at 64 sites in North Carolina and 42 sites in South Carolina. The authors reported reductions of 5.1% for total crashes, 13.3% for angle crashes, and 10.2% for fatal and injury crashes. Additionally, flashing beacons were found to be most effective in rural areas and at four-way stop-controlled intersections. A more recent study evaluated 74 stop-controlled intersections in North Carolina (Simpson and Troy 2013). The study focused on Vehicle Entering When Flashing signs. The results of the study showed that the signs were most effective at two-lane stop-controlled intersections, which saw a reduction in total crashes of 25%.

When available in the literature, the cost estimates of safety treatments were noted during this study. Quotes from equipment vendors were also sought to supplement the cost information. These estimates are included in Appendix A.

5. DESIGN GUIDANCE FOR J-TURNS

5.1 Crash Analysis

J-turn crash reports were reviewed to identify patterns in crashes. Data were collected for the period after the J-turns were in operation. The crash information was then used to develop crash diagrams illustrating different crash types. This section discusses sampling, site characteristics, crash data collection, and crash type analysis.

The master list of J-turns in Missouri used for this study consisted of 18 facilities that were in operation at the time of this research. The criteria used for selecting sites for detailed collision diagram analysis consisted of crash data availability, configuration of the pre-J-turn intersection, lack of influence from other facilities, and no significant geometric or other changes during the post-J-turn analysis period. Twelve of the eighteen facilities satisfied the site selection criteria. These twelve facilities are listed in Table 5.1.

Table 5.1. J-turn facilities selected

J-turn	City	Location	Open	Distance (ft)	
				U-turn 1	U-turn 2
1	Imperial	RT M and Old Lemay Ferry Connector	Sep-07	800	1,900
2	Byrnes Mill	MO 30 and Upper Byrnes Mill Road	Dec-12	1,500	1,700
3	Jefferson City	US 54 and Honey Creek Road	Nov-11	1,900	1,900
4	Jefferson City	US 54 and Route E	Oct-11	1,700	N/A
5	Columbia	US 63 and Route AB	Nov-12	2,300	3,000
6	Columbia	US 63 and Bonne Femme Church Road	Nov-12	900	1,400
7	Osceola	MO 13 and Old MO 13/364 E	Jul-09	1,100	980
8	Ridgedale	US 65 and Rochester Road	Dec-12	730	990
9	Sheridan	US 65 and MO 215/ RT O	Nov-09	630	630
10	Jackson	US 65 and MO 38	Nov-09	630	630
11	Jackson	US 65 and Ash Street/ Red Top Road	Nov-09	630	630
12	Sheridan	US 65 and RT AA	Nov-09	650	1,300

Additional site characteristics, including urban/rural classification and major and minor road AADTs, are presented in Table 5.2.

Table 5.2. Designation area and AADT counts

J-turn	Location	Area*	AADT Major Road	AADT Minor Road
1	RT M and Old Lemay Ferry Connector	Urban	9,320**	358
2	MO 30 and Upper Byrnes Mill Road	Urban	23,091	2,226
3	US 54 and Honey Creek Road	Rural	18,213	435
4	US 54 and Route E	Rural	15,097	1,017
5	US 63 and Route AB	Rural	26,956	1,020
6	US 63 and Bonne Femme Church Road	Urban	26,388	1,504
7	MO 13 and Old MO 13/364 E	Rural	11,109	467
8	US 65 and Rochester Road	Rural	11,584	486
9	US 65 and MO 215/ RT O	Rural	7,573	982
10	US 65 and MO 38	Rural	6,975	822
11	US 65 and Ash Street/ Red Top Road	Rural	6,631	524
12	US 65 and RT AA	Rural	9,407	932

* A rural area has less than 5,000 population; otherwise, the area is urban

** AADT for the year 2013

Satellite images and distances between the minor road and the U-turn are also provided for each site in the following summaries.

RT M and Old Lemay Ferry Connector. This facility is a three-leg intersection with two U-turns. The U-turn to the east is at 1,900 ft and the U-turn to the west is at 800 ft from the minor road. Left turns from the major road are not allowed at the intersection. Figure 5.1 shows the aerial image of the facility.



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Figure 5.1. RT M and Old Lemay Ferry Connector aerial image

MO 30 and Upper Byrnes Mill Road. This facility is a four-leg intersection with two U-turns. The U-turn to the east is at 1,500 ft and the U-turn to the west is at 1,700 ft from the minor road. There is a median opening to allow left turns from the major road to turn at the intersection. Figure 5.2 shows the aerial image of the facility.



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Figure 5.2. MO 30 and Upper Byrnes Mill Road aerial image

US 54 and Honey Creek Road. This facility is a four-leg intersection with two U-turns. The U-turns are both at a distance of 1,900 ft from the minor road. There is a median opening to allow left turns from the major road to turn at the intersection. Figure 5.3 shows the aerial image of the facility.



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Figure 5.3. US 54 and Honey Creek Road aerial image

US 54 and Route E. This facility is a four-leg intersection with only one U-turn. The U-turn east of the minor road is at a distance of 1,700 ft. There is a median opening to allow left turns from the major road to turn at the intersection. Figure 5.4 shows the aerial image of the facility.



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Figure 5.4. US 54 and Route E aerial image

US 63 and Route AB. This facility is a four-leg intersection with two U-turns. The U-turn to the right (north) is at a distance of 3,000 ft and the U-turn to the left (south) is at 2,300 ft from the minor road. Left turns from the major road are not allowed at the intersection. Figure 5.5 shows the aerial image of the facility.



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Figure 5.5. US 63 and Route AB aerial image

US 63 and Bonne Femme Church Road. This facility has two four-leg intersections between the U-turns. The U-turn to the right (north) is at a distance of 1,400 ft and the U-turn to the left (south) is at 900 ft to the closest minor road access. Left turns from the major road are not allowed at the intersection. Figure 5.6 shows the aerial image of the facility.



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Figure 5.6. US 63 and Bonne Femme Church Road aerial image

MO 13 and Old MO 13/364 E. This facility is a four-leg intersection with two U-turns. The U-turn to the right (north) is at a distance of 980 ft and the U-turn to the left (south) is at 1,100 ft from the minor road. There is a median opening to allow left turns from the major road to turn at the intersection. The U-turns have additional islands to facilitate turning movements by larger vehicles. Figure 5.7 shows the aerial image of the facility.



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Figure 5.7. MO 13 and Old MO 13/364 E aerial image

US 65 and Rochester Road. This facility is a four-leg intersection with two U-turns. The U-turn to the right (north) is at a distance of 990 ft and the U-turn to the left (south) is at 730 ft from the minor road. There is a median opening to allow left turns from the major road to turn at the intersection. The U-turns have additional islands to facilitate turning movements by larger vehicles. Figure 5.8 shows the aerial image of the facility.



©2016 Google, Image USDA Farm Service Agency

Figure 5.8. US 65 and Rochester Road aerial image

US 65 and MO 215/ RT O. This facility is a four-leg intersection with two U-turns. The U-turns are both at a distance of 630 ft from the minor road. Left turns from the major road are not allowed at the intersection. There are additional islands to facilitate turning movements by larger vehicles. Figure 5.9 shows the aerial image of the facility.



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Figure 5.9. US 65 and MO 215/ RT O aerial image

US 65 and MO 38. The facility is a four-leg intersection with two U-turns. The U-turns are both at a distance of 630 ft from the minor road. Left turns from the major road are not allowed at the intersection. There are additional islands to facilitate turning movements by larger vehicles. Figure 5.10 shows the aerial image of the facility.



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Figure 5.10. US 65 and MO 38 aerial image

US 65 and Ash Street/ Red Top Road. This facility is a four-leg intersection with two U-turns. The U-turns are both at a distance of 630 ft from the minor road. Left turns from the major road are not allowed at the intersection. There are additional islands to facilitate turning movements by larger vehicles. Figure 5.11 shows the aerial image of the facility.



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Figure 5.11. US 65 and Ash Street/Red Top Road aerial image

US 65 and RT AA. This facility is a four-leg intersection with two U-turns. The U-turn to the right (north) is at 1,300 ft and the U-turn to the left (south) is at 650 ft from the minor road. Left turns from the major road are not allowed at the intersection. The U-turns have islands to facilitate turning movements by larger vehicles. Figure 5.12 shows the aerial image of the facility.



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Figure 5.12. US 65 and RT AA aerial image

5.1.1 Crash Data Collection

Crash data were collected for the entire footprint of the J-turn (U-turn to U-turn) and additional areas of influence. The influence area upstream of the U-turn included the area where mainline traffic is influenced by vehicles coming out of the U-turn. The influence areas consisted of 1,000 ft beyond the U-turn in each direction for the major road and 250 ft of the minor road. Crashes were queried using the Accident Browser application in the MoDOT Transportation Management System. The period of analysis for each facility was from the date the facility opened to traffic with the J-turn geometric design until the end of 2014. A total of 183 crashes occurred at all facilities within the extended J-turn footprint. All 183 crash reports were manually reviewed and located on a generic J-turn design layout in AutoCAD. Figure 5.13. shows an example of the crash locations for the J-turn at the intersection of RT M and Old Lemay Ferry Connector.

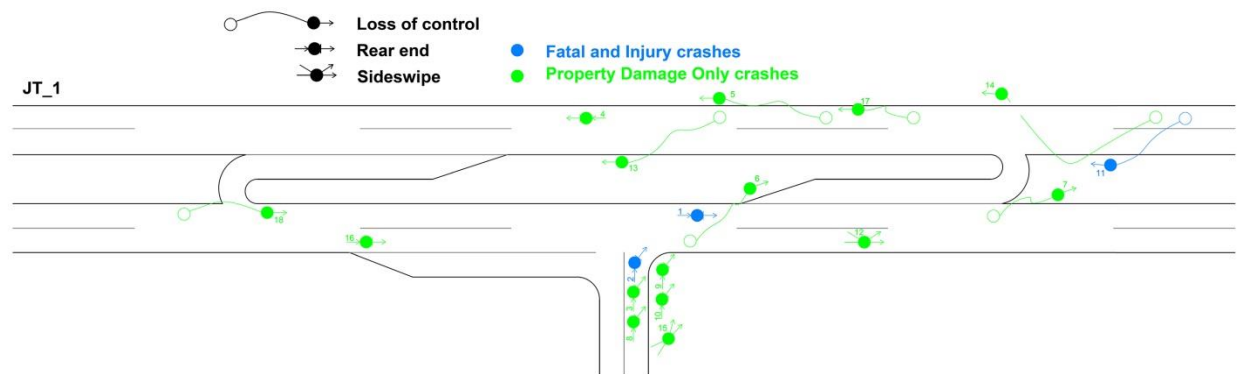


Figure 5.13. Crash locations at RT M and Old Lemay Ferry Connector

The located crashes were further filtered based on whether they were related to the J-turn. For example, crashes that occurred due to inclement weather, impaired driving, and other non-J-turn-related circumstances were not included in further analysis. This was done to eliminate any non-J-turn-related factors that may have contributed to the crashes. Thus, the remaining crashes occurred due to the geometry and/or operations of the J-turn design.

5.1.2 Collision Diagram Analysis

A collision diagram analysis helped to identify crashes according to the location and geometry of the J-turn. A total of 57 crashes were attributed to the J-turns. These crashes were separated into five types: (1) major road sideswipe, (2) major road rear-end, (3) minor road rear-end, (4) loss of control, and (5) merging from U-turn. Figure 5.14 shows the results of the collision diagram analysis, and Table 5.3 shows the percentage of each crash type.

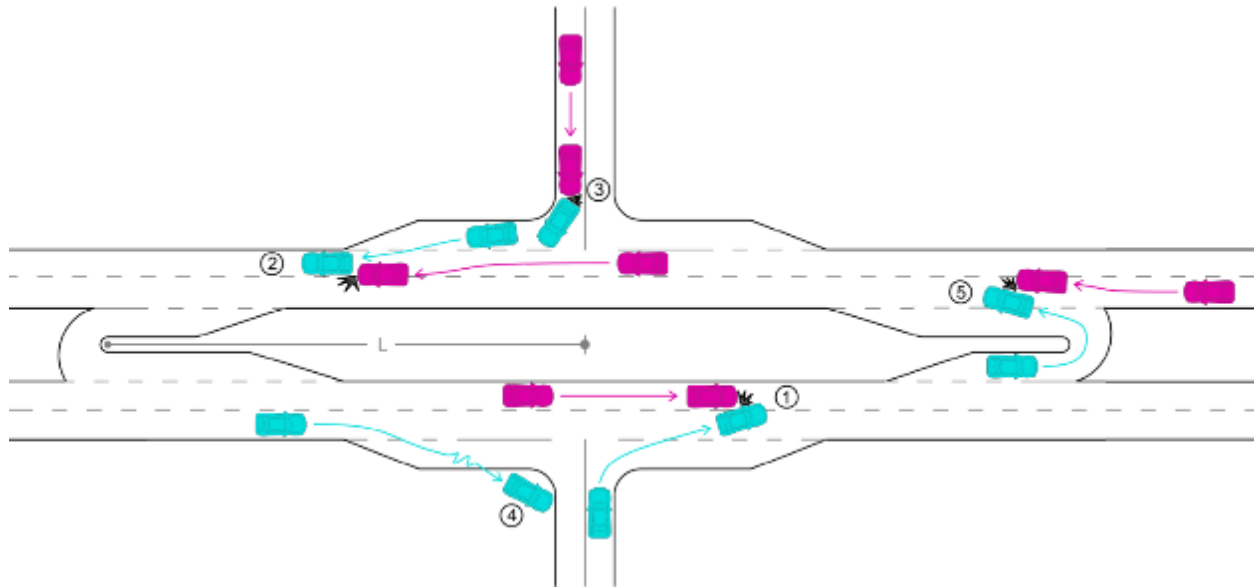


Figure 5.14. Results of collision diagram analysis

Table 5.3. Types of J-turn crashes

Type of Crash	Crashes (#)	Crashes (%)
Major road sideswipe	18	31.6%
Major road rear-end	16	28.1%
Minor road rear-end	9	15.8%
Loss of control	8	14.0%
Merging from U-turn	6	10.5%

The most frequent crashes at the J-turns were sideswipe (31.6%) and rear-end (28.1%) on the major road. Most of these crashes occurred while vehicles were merging with traffic or changing lanes to enter the U-turn. High speed differential and driver inattention were common circumstances in most of the crashes that occurred at the J-turn facilities.

The rear-end crashes on minor roads occurred when drivers were unable to stop in time and collided with a vehicle ahead that suddenly stopped or slowed down to look for a gap in the through traffic on the major road. Most of the loss of control crashes occurred due to driver intention, improper lane use, or high speeds and occurred in deceleration lanes. For the top two crash types, sideswipes and rear-end crashes on the major road, crash rates were computed as a function of traffic exposure and segment length as follows:

$$\text{Crashes per Million Vehicle Miles Traveled (MVMT)} = \frac{A \times 1,000,000}{L \times \text{AADT} \times 365} \quad 5.1$$

where,

- A is the average number of crashes per year

- L is the segment length (miles)
- $AADT$ is the total entering vehicles per year

Figure 5.15 presents the crash rates categorized by the distance between the minor road and the U-turn.

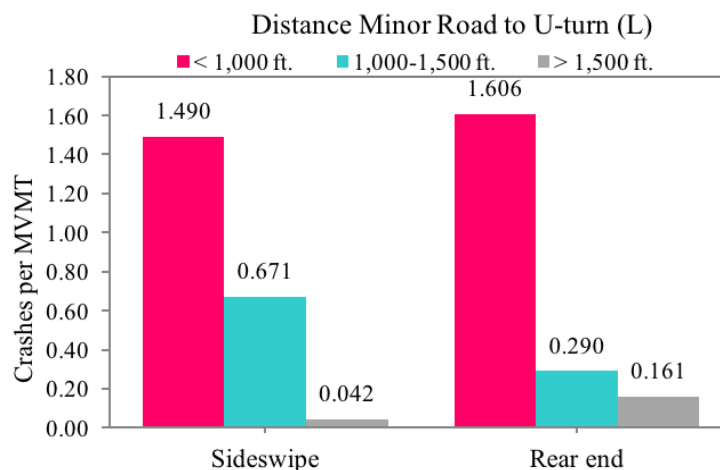


Figure 5.15. Sideswipe and rear-end crash rates on the major road

For both sideswipe and rear-end crashes, crash rates decreased as the distance from the minor road to the U-turn increased. The longer distance allows merging vehicles to reach major road operating speeds, thus making it safer for merging vehicles to follow other vehicles in the lane and to make lane changes. J-turn sites with a spacing of 1,500 ft or greater experienced the lowest crash rates.

5.2 Simulation Analysis

5.2.1 Simulation Model Development

Microsimulation was used to analyze the safety effects of two different J-turn design considerations: presence or absence of acceleration lanes and the distance between the minor road and the U-turn. The simulation model used in this research is derived from field data collected in a previous MoDOT research project from 2014 (Edara et al. 2014). The previous J-turn field site is located near Deer Park Road on Highway 63, south of Columbia, Missouri. This section of Highway 63 is a rural four-lane highway with a speed limit of 70 mph. This segment consists mainly of tangents with no sharp horizontal curves or steep vertical grades. The satellite image and the corresponding Vissim simulation model layout are shown in Figure 5.16.



©2016 Google, Image Landsat/Copernicus

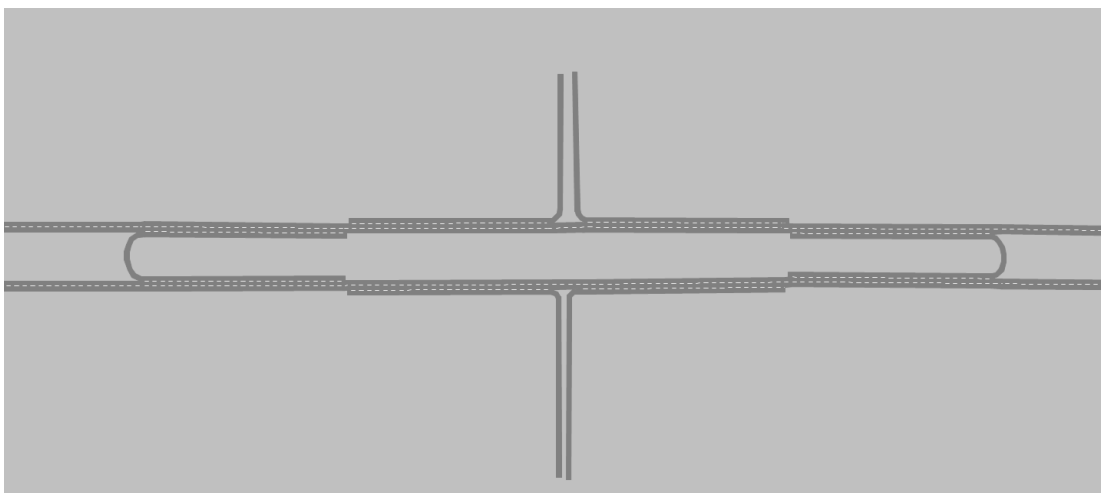


Figure 5.16. Satellite image and Vissim simulation model layout of a J-turn on Highway 63 at Deer Park Road: satellite image (top) and simulation layout (bottom)

For the distance between the minor road and the U-turn, three distances were analyzed: 1,000 ft, 2,000 ft, and 3,000 ft. For the presence or absence of acceleration lanes, two different layouts were analyzed, as shown in Figure 5.17.

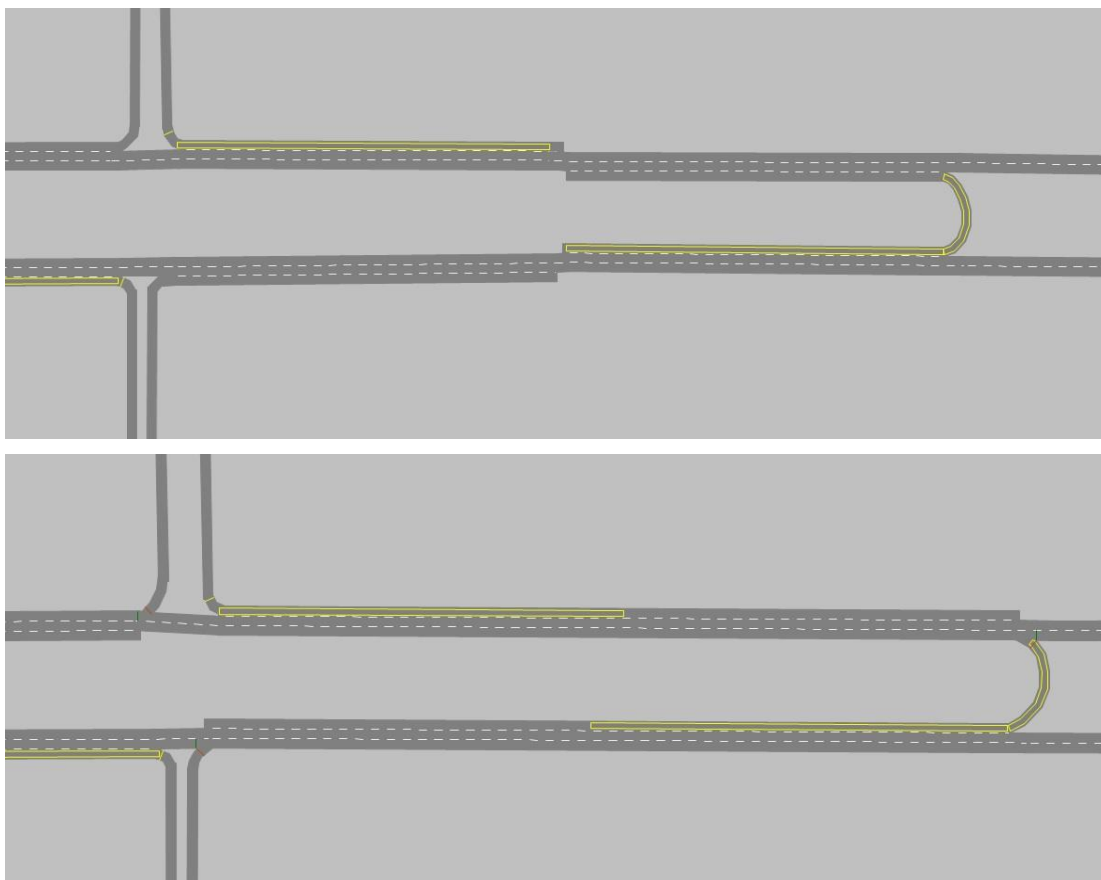


Figure 5.17. J-turn layouts with (top) and without (bottom) an acceleration lane

The first layout (Figure 5.17, top) includes an acceleration lane extending from the minor road to half the distance to the U-turn and a deceleration lane for the U-turn starting at the end of the acceleration lane and extending to the U-turn. In the other direction, an acceleration lane is provided for vehicles merging onto major road from the U-turn lane and a deceleration lane is provided for vehicles exiting onto the minor road. The second layout (Figure 5.17, bottom) does not contain an acceleration lane for minor road traffic or for U-turn traffic. The deceleration lane extends the entire length between the U-turn and the minor road. These two layouts were recommended by the project's technical advisory panel, which included MoDOT safety engineers.

Several parameters in Vissim were optimized in order to accurately simulate vehicles at a J-turn. These parameters included reduced speed areas (length and magnitude), desired speed decisions, and lane change distance upstream of a connector. For example, Figure 5.18 shows the lane change distance parameter window in Vissim.

Connector

No.: 10008 Name:

Behavior type: 6: J-turn test

Display Type: 1: Road gray

from link: to link:

No.: 7 No.: 10

At: 491.678 ft At: 5.762 ft

Lane 1
Lane 2
Lane 3

Lane 1
Lane 2
Lane 3

Length: 132.344 ft

Spline: 7

Lane Change Display Dyn. Assignment Other

Count	Index	BlockedV	NoLnChL	NoLnChR	NoLnChLV	NoLnChRV
1	1		<input checked="" type="checkbox"/>	<input checked="" type="checkbox"/>		

Route

Emergency Stop: 16.4 ft back

Lane change: 1000.0 ft back ☐ per lane

Desired Direction

☒ All ☐ Right ☐ Left

OK Cancel

Figure 5.18. Connector tab from Vissim, showing the lane change distance parameter

The lane change distance parameter specifies the distance upstream from a connector where vehicles start to look for lane changing gaps to stay on their desired path. The value of this parameter was based on trial and error through manual observation of the simulations. The value was different for the two layouts.

The calibration procedure in this study used disaggregated data of individual vehicle speeds measured in the field as part of a previous project (Edara et al. 2014). Thus, the calibration procedure was more robust than the state of the practice, which relies on aggregated sensor speeds on a roadway. A map showing the placement of the field data collection equipment used in Edara et al. (2014) is provided in Figure 5.19.



Edara et al. 2014, Missouri DOT, Imagery ©2013 Digital Globe/USDA Farm Service Agency, Map data ©2013 Google

Figure 5.19. Data collection equipment used in Edara et al. 2014

Several cameras and radar guns were used to extract traffic volumes and vehicle speeds (see Figure 5.20). The a.m. peak period data were collected in the southbound direction, and the p.m. peak period data were collected in the northbound direction.



Figure 5.20. Radar speed gun view

The speed distributions of merging vehicles from the minor road (Route E) and through traffic on the major road are shown in Figures 5.21 and 5.22, respectively. The 85th percentile speeds were 75 mph and 70 mph for passenger cars and trucks, respectively, on the major road and 64 mph for merging vehicles from the minor road.

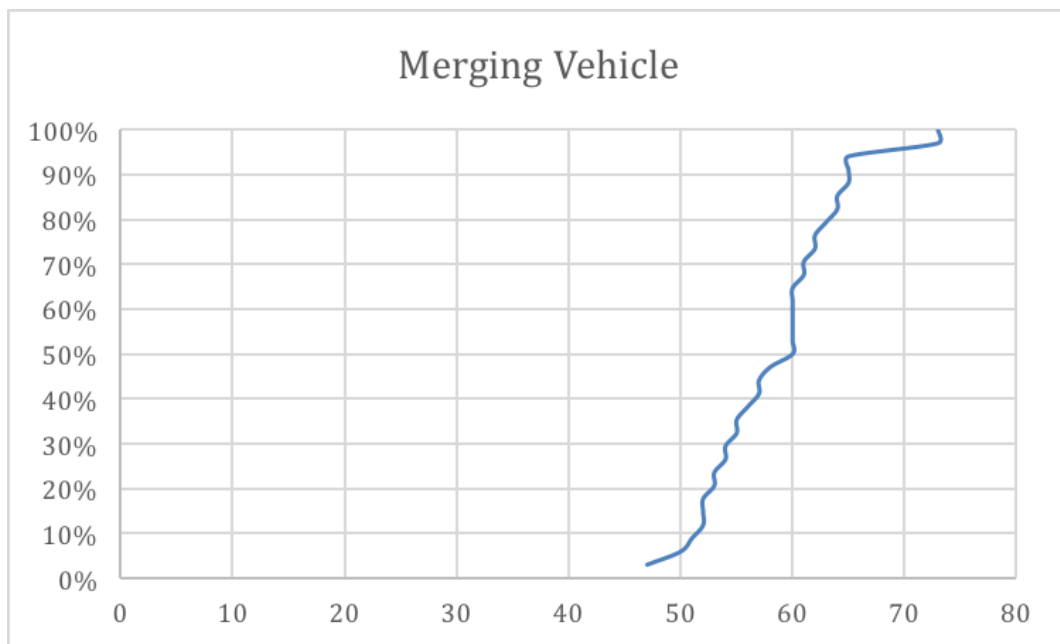


Figure 5.21. Merging vehicle speed distribution

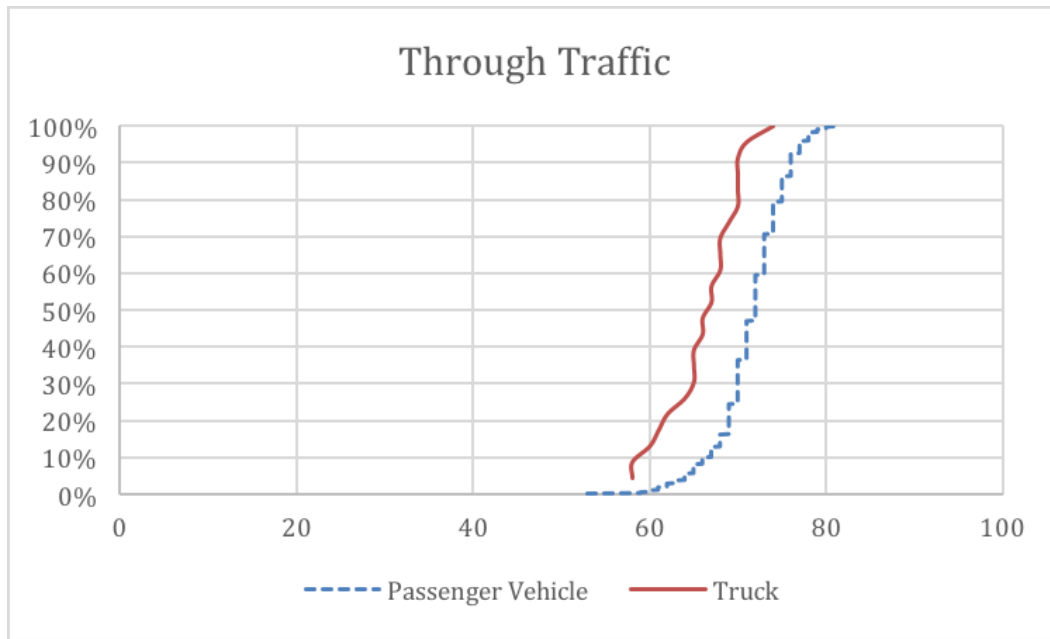


Figure 5.22. Through traffic speed distribution

These speed distributions were then defined in Vissim using the desired speed distribution parameter windows, as shown in Figure 5.23.

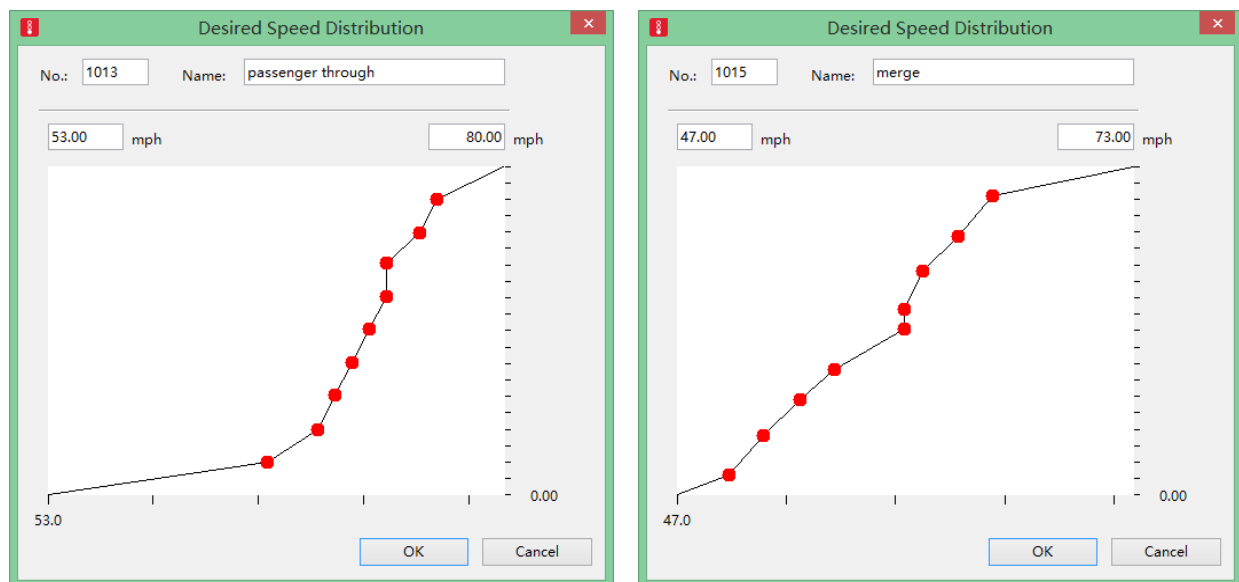


Figure 5.23. Desired speed distributions in Vissim: passenger through (left) and merging/diverging (right)

Different volume scenarios were generated for analyzing the performance of the J-turn. Table 5.4 shows the base volume scenario used, measured in vehicles per hour (veh/hr).

Table 5.4. Base condition major and minor road flow rates

No.	Movement	Diagram	Veh/hr	Total
1	Major road through	→	1,443	
2	Major road left turn	↵	18	1,504
3	Major road right turn	↘	43	
4	Minor road through	↑	22	
5	Minor road left turn	↶	16	308
6	Minor road right turn	↷	270	

The major road volumes shown in Table 5.4 were obtained from the field data described above. The field-observed minor road volumes were low and did not generate enough conflicts to be useful for safety analysis. Therefore, higher values were used.

The base case only shows 1 of the 12 volume scenarios that were studied for this project. Table 5.5 shows all 12 major and minor road flow combinations.

Table 5.5. Volume scenarios

No.	Major Road Total (veh/hr)	Minor Road Crossing (veh/hr)	Minor Road Right Turn (veh/hr)	Total Minor/Major Ratio
1	1,000	150	150	30%
2	1,000	250	250	50%
3	1,000	350	350	70%
4	1,300	195	195	30%
5	1,300	325	325	50%
6	1,300	455	455	70%
7	1,504	226	226	30%
8	1,504	376	376	50%
9	1,504	526	526	70%
10	1,800	270	270	30%
11	1,800	450	450	50%
12	1,800	630	630	70%

The Minor Road Crossing column in Table 5.5 includes both minor road left turns and minor road through movements. The volume scenarios ranged from low to high volumes. These 12

volume scenarios were then studied for the three U-turn distances of 1,000, 2,000, and 3,000 ft and for the presence/absence of an acceleration lane, thus resulting in a total of 72 combinations.

The FHWA's SSAM includes an option where unrealistic conflicts (e.g., time to collision = 0) can be filtered from the output. The SSAM user manual provides guidance on selecting the threshold values for the filters (Gettman and Head 2003). Figure 5.24 shows the filters used in this study for all volume and design scenarios.

3k_medium flow_no accel

Filter Surrogate Thresholds

0.05	<=	TTC	<=	1.5	seconds
0	<=	PET	<=	4.8	seconds
1.467	<=	MaxS	<=	35.75	meters/second
0.022	<=	DeltaS	<=	45.467	meters/second
-7.765	<=	DR	<=	2.906	meters/second ²
-8.145	<=	MaxD	<=	2.906	meters/second ²
0.011	<=	MaxDeltaV	<=	32.694	meters/second

Filter Conflict Types

Include Conflict Types: unclassified
crossing
rear end
lane change
ALL

Filter Links

Include Links: ALL
1
2

Filter TRJ Files

Include TRJ Files: D:\Google Drive\1 J turn\simulation\J-turn medium flo
D:\Google Drive\1 J turn\simulation\J-turn medium flo
D:\Google Drive\1 J turn\simulation\J-turn medium flo
D:\Google Drive\1 J turn\simulation\J-turn medium flo
D:\Google Drive\1 J turn\simulation\J-turn medium flo

Filter Area

Lower Left (x,y): -814 -382

Upper Right(x,y): 1,533 217

Apply Restore Defaults

Configuration Conflicts Summary Filter Ttest Map

Figure 5.24. Applied SSAM filters used in the conflict analysis

5.2.2 Simulation Results

5.2.2.1 Designs with Acceleration Lanes

The four charts in Figure 5.25 show the average conflicts registered by SSAM for all 12 volume combinations, grouped by major road volume.

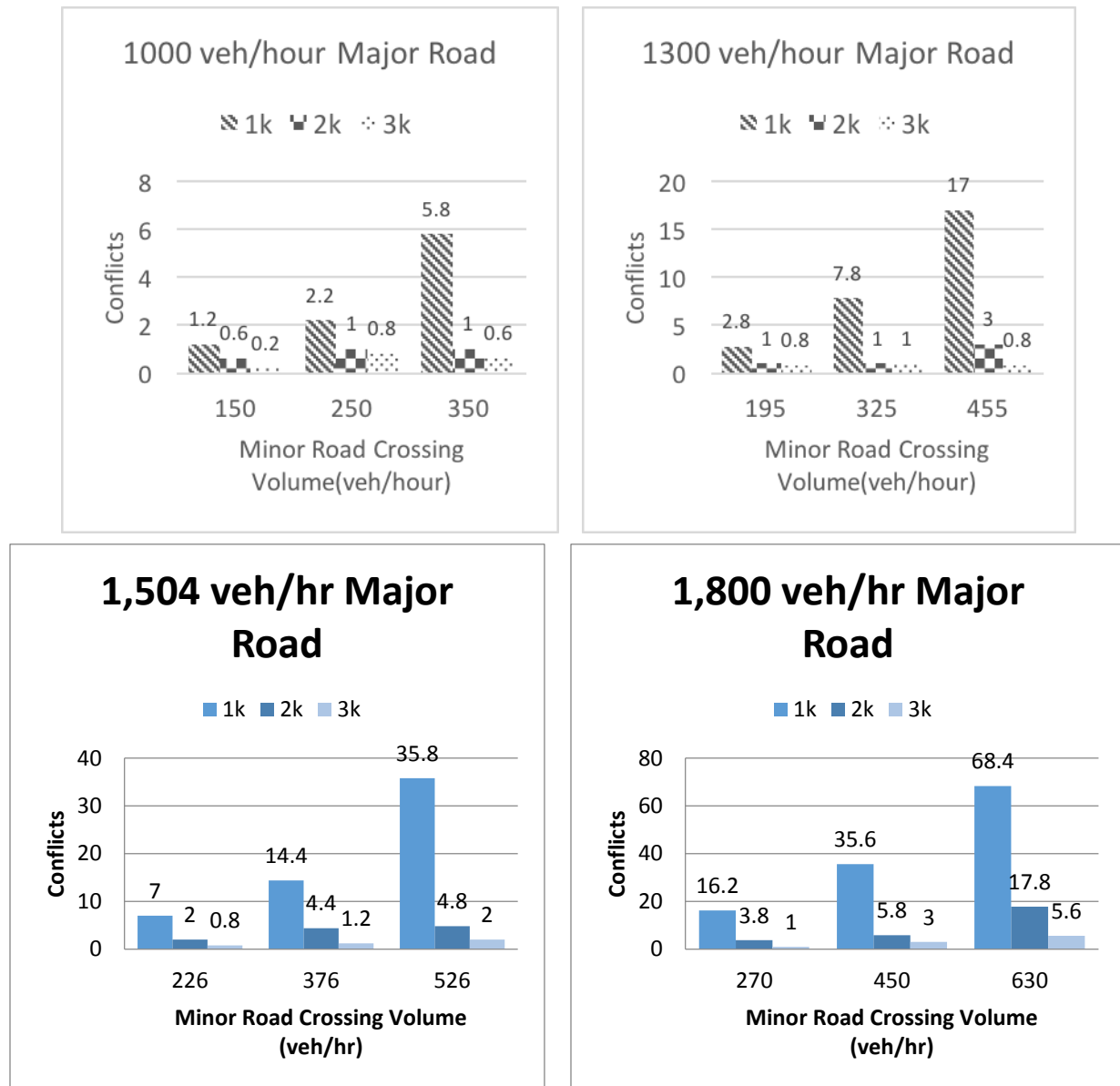


Figure 5.25. Conflict counts for designs with an acceleration lane: 1,000 veh/hr major road (top left), 1,300 veh/hr major road (top right), 1,504 veh/hr major road (bottom left), 1,800 veh/hr major road (bottom right)

In each chart, the x axis stands for the minor road crossing volume and the y axis stands for the conflicts. In addition to the crossing volume, an equal number of right turning vehicles was also simulated. For example, the total minor road volume for the scenario with a crossing volume of 150 veh/hr was 300 veh/hr. Each scenario was run five times using different random seeds in Vissim, and the results were averaged across the five runs. Striped bars in the charts in Figure 5.25 indicate 1,000 ft (1k) spacing, squared bars indicate 2,000 ft (2k) spacing, and dotted bars indicate 3,000 ft (3k) spacing.

The results were consistent for all volume scenarios. The number of conflicts decreased with an increase in the spacing between the minor road and the U-turn. For example, the lowest volume combination, 1,000 veh/hr total on the major road and 150 veh/hr on the U-turn, witnessed 1.2 conflicts for the 1,000 ft spacing, 0.6 for the 2,000 ft spacing, and 0.2 for the 3,000 ft spacing. This effect is more significant when the traffic volume is higher. For the highest volume scenario of 1,800 veh/hr on the major road and 630 veh/hr on the U-turn, the number of conflicts dropped from 68.4 to 17.8 for the 2,000 ft spacing, a difference of 50.6, and to merely 5.6 for the 3,000 ft spacing.

Although it is clear from the results that longer spacing values decreased the number of conflicts, the reduction in conflicts is not linear. For example, the second heaviest volume combination resulted in a reduction of 31 conflicts from the 1,000 ft to the 2,000 ft spacing and merely a reduction of 2.8 conflicts from the 2,000 ft to the 3,000 ft spacing. Thus, a spacing of 2,000 ft may be sufficient to provide a good trade-off between safety and cost-effective J-turn design.

5.2.2.2 Designs without Acceleration Lanes

In general, the lack of an acceleration lane increased the queuing on the minor road for vehicles waiting for a gap to merge onto the major road. The numbers of conflicts for designs without an acceleration lane are shown in the charts in Figure 5.26.

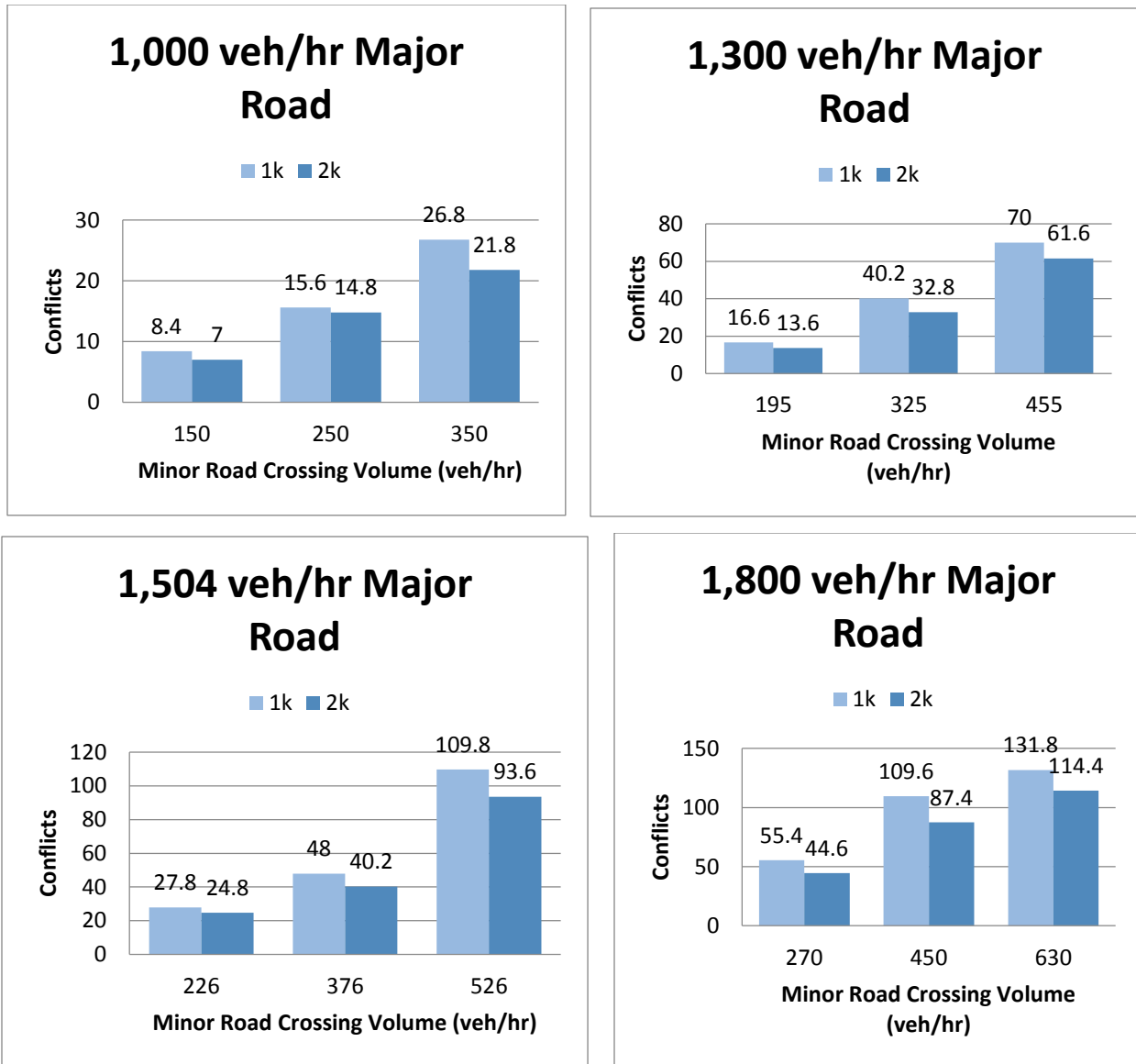


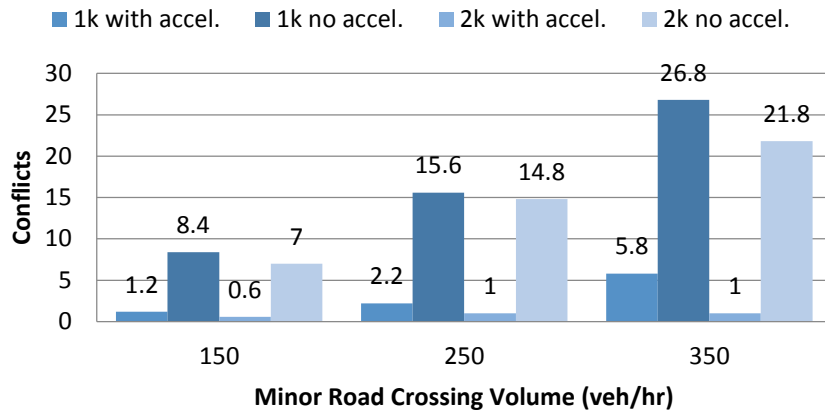
Figure 5.26. Conflict counts for designs without an acceleration lane: 1,000 veh/hr major road (top left), 1,300 veh/hr major road (top right), 1,504 veh/hr major road (bottom left), 1,800 veh/hr major road (bottom right)

Due to the lack of acceleration lanes, only two spacing combinations were evaluated: 1,000 ft and 2,000 ft. Overall, the number of conflicts decreased when the spacing increased from 1,000 ft to 2,000 ft. For example, in Figure 5.26 (top right) conflicts dropped from 16.6 to 13.6 for 195 U-turn vehicles, 40.2 to 32.8 for 325 U-turn vehicles, and 70 to 61.6 for 455 U-turn vehicles.

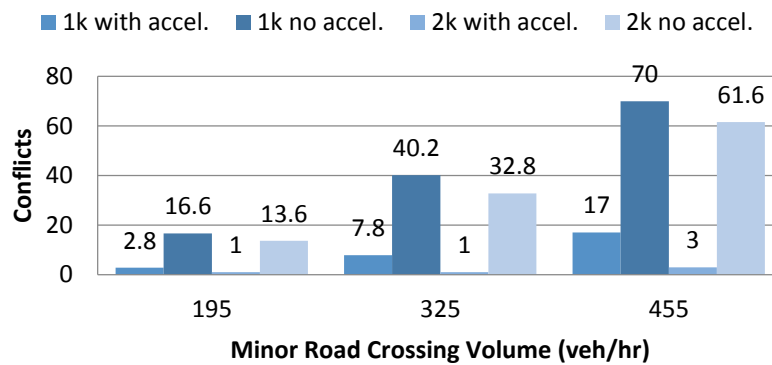
5.2.2.3 Comparison of Designs With and Without Acceleration Lanes

The charts in Figure 5.27 show the numbers of conflicts for designs with and without acceleration lanes across all volume scenarios.

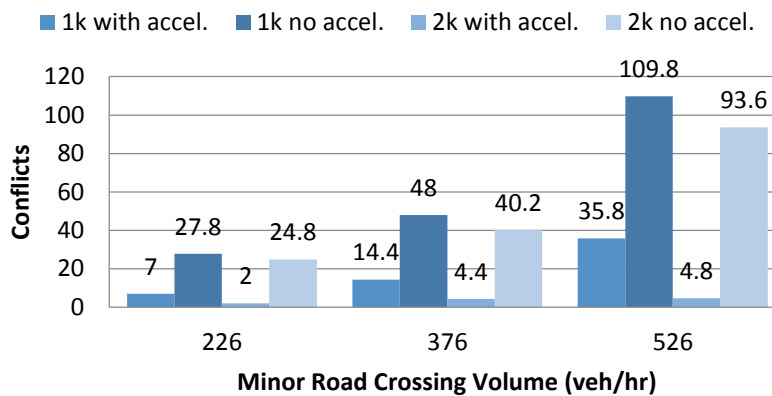
1,000 veh/hr Major Road



1,300 veh/hr Major Road



1,504 veh/hr Major Road



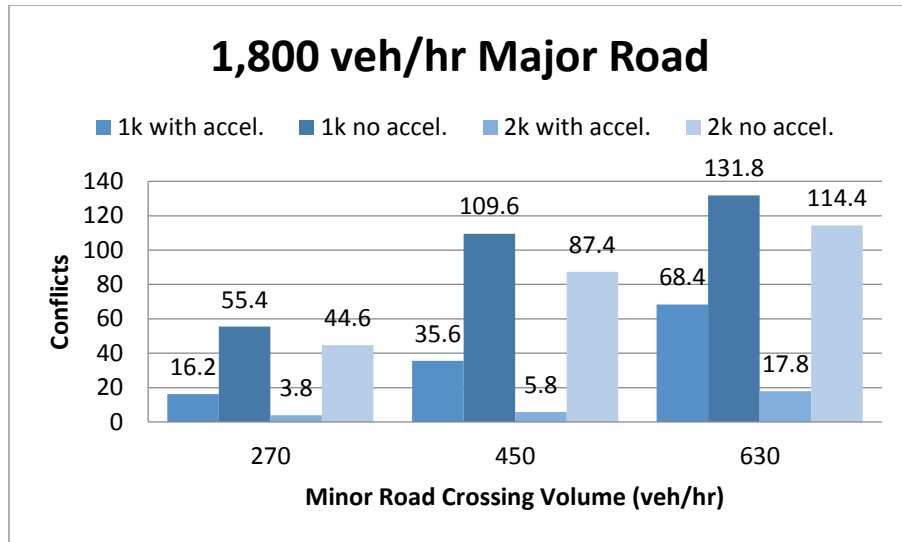


Figure 5.27. Comparison of conflict counts for designs with and without an acceleration lane

In the charts in Figure 5.27, striped bars represent the designs with acceleration lanes and checkered bars represent the designs without acceleration lanes. For each volume combination and the same U-turn spacing, the designs without an acceleration lane experienced more conflicts than the designs with an acceleration lane. Therefore, acceleration lanes resulted in better safety for all spacing and volume combinations.

One goal of this project was to determine the optimal spacing between the U-turn and the minor road for different volume and design combinations. Table 5.6, which shows the recommended minimum spacing for each volume and design scenario, was compiled based on the results of the conflict measures generated from the simulation analysis.

Table 5.6. Recommended minimum spacing for each scenario

Major Total (veh/hr)	Minor Crossing (left and through) (veh/hr)	Minor Crossing (right) (veh/hr)	Total Minor/Major	With Acceleration Lane (ft)	No Acceleration Lane (ft)
1,000	150	150	30%	1,000-2,000	1,000
	250	250	50%	1,000-2,000	1,000
	350	350	70%	2000	1,000-2,000
1,300	195	195	30%	1,000-2,000	1,000
	325	325	50%	2000	1,000-2,000
	455	455	70%	2,000-3,000	1,000-2,000
1,504	226	226	30%	2,000	1,000
	376	376	50%	>2,000	1,000-2,000
	526	526	70%	>2,000	1,000-2,000
1,800	270	270	30%	2,000	1,000
	450	450	50%	2,000	2,000
	630	630	70%	3,000	2,000

As previously concluded, acceleration lanes were found to be safer for all volume combinations studied in this project. When acceleration lanes can be provided, the recommended spacing is lower for low-volume combinations, as shown in the With Acceleration Lane column in Table 5.6. If acceleration lanes cannot be provided, the No Acceleration Lane column in Table 5.6 provides guidance on the minimum spacing recommended for the different volume combinations. The minimum spacing values for the No Acceleration Lane column should not be compared with the values for the With Acceleration Lane column. For example, the row corresponding to a major road volume of 1,504 veh/hr and a minor road crossing volume for left turn and through vehicles of 226 veh/hr shows that 2,000 ft is recommended when an acceleration lane can be provided and 1,000 ft when an acceleration lane cannot be provided. This recommendation, however, does not mean that the absence of an acceleration lane results in lower spacing; the chart in Figure 5.27 for 1,504 veh/hr clearly shows fewer conflicts when an acceleration lane is provided. Therefore, the values in the No Acceleration Lane column must only be interpreted for the various combinations when an acceleration lane is absent.

6. CONCLUSIONS

System-wide safety treatments are aimed at treating select types of crashes occurring across a state. In Missouri, cable median barriers and shoulder line rumble strips are examples of successful system-wide safety treatments that have been deployed across the state to reduce lane departure fatalities. Missouri's Strategic Highway Safety Plan established a short-term goal of reducing traffic fatalities to 700 per year by 2016 as an intermediate step towards the long-term goal of zero roadway deaths in the state. This project synthesized the existing state of the practice related to system-wide treatments, specifically those that have not previously been implemented in Missouri. The synthesis covered three areas: (1) horizontal curves, (2) intersections, and (3) wrong-way crashes. The identified safety treatments work in conjunction with the "Necessary Nine" strategies identified in the Missouri Blueprint. The safety effectiveness, implementation guidelines, limitations, costs, and concerns of the treatments were documented. The synthesis can assist MoDOT in selecting system-wide treatments for future deployment in the state.

Countermeasures related to signage, design, ITS, and drivers were reviewed to address wrong-way crashes. Innovative signage strategies including lowering the height of signs, deploying oversized signs, providing illumination, and doubling the number of signs are low-cost solutions that can be deployed system-wide. Design countermeasures such as avoiding left-side exit ramps, using raised medians on crossroads, and improving sight distance are also recommended. ITS technology options are more expensive and therefore may not be suitable for system-wide deployment. Detection and alert systems based on video radar or in-pavement sensors have been piloted in a few states.

Countermeasures targeting horizontal curve crashes may involve augmenting the minimum recommended MUTCD signs and devices at horizontal curves. These countermeasures include improved curve signing through the use of additional chevrons, flashing beacons at sharp curves, dynamic curve guidance systems, and dynamic speed warning systems. Pavement marking treatments such as speed reduction markings, warning symbols painted on the pavement, and high-friction pavement treatments are recommended for system-wide deployment in Missouri. MoDOT has successfully utilized two pavement marking treatments in the past: wider edge lines and rumble strips/stripes.

Treatments to enhance signalized intersection safety include increasing clearance intervals, changing left turns from permissive to protected-permissive, installing flashing yellow arrows, providing dynamic signal warnings, installing red light cameras, and improving signal visibility. Based on the safety effectiveness reported in the literature, providing dynamic signal warnings and improving signal visibility are recommended for future consideration as system-wide treatments at signalized intersections in Missouri.

A detailed analysis of the collision diagrams for crashes that occurred at 12 J-turn sites in Missouri revealed the proportion of crash types that occurred at these sites. The five crash types are (1) major road sideswipe (31.6%), (2) major road rear-end (28.1%), (3) minor road rear-end (15.8%), (4) loss of control (14%), and (5) merging from U-turn (10.5%). Most of the major road

sideswipe and rear-end crashes occurred while vehicles were merging with traffic or changing lanes to enter the U-turn. Higher speed differentials between merging and major road vehicles and driver inattention were common factors in most crashes that occurred at the J-turn facilities. The crash rates computed from the collision diagram analysis showed that crash rates for both sideswipe and rear-end crashes decreased with an increase in the spacing between the minor road and the U-turn. The longer spacing allowed merging vehicles to reach major road operating speeds, thus making it safer to follow other vehicles in the lane and to make lane changes. J-turns with a spacing of 1,500 ft or greater experienced the lowest crash rates.

A simulation analysis was conducted to further study the impact of different design variables on the safety of J-turns. Specifically, the effect of the presence of an acceleration lane and the spacing from the minor road to the U-turn were investigated. A base simulation model was created and calibrated using field data collected during a previous MoDOT project on J-turns. The calibrated model was then used to study various combinations of major road and minor road volumes and design variables. The simulation analysis helped in the development of guidance on the recommended spacing for various major road and minor road volume scenarios. For all of the studied scenarios, the presence of an acceleration lane resulted in significantly fewer conflicts. Therefore, acceleration lanes are recommended for all J-turn designs, including lower volume sites. Additionally, while spacing between 1,000 ft and 2,000 ft was found to be sufficient for low-volume combinations, a spacing of 2,000 ft is recommended for medium- to high-volume conditions.

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APPENDIX. COUNTERMEASURE EFFECTIVENESS AND COST

A.1 Wrong-Way Crashes

A.1.1 Ramp Terminals

Countermeasure	Description	Estimated Effectiveness	Effect on Crash Frequency	Estimated Cost	Actual Cost
Minimum signing	One 36"x12" (R6-1), one 24"x24" (R5-1), and one 30"x18" (R5a-1)	Low	—	Low	\$101 per site (TAPCO 2015)
	One 54"x18" (R6-1), one 30"x30" (R5-1), and one 36"x24" (R5a-1)	Low	—	Low	\$167 per site (TAPCO 2015)
	One 54"x18" (R6-1), one 36"x36" (R5-1), and one 42"x30" (R5a-1)	Low	—	Low	\$197 per site (TAPCO 2015)
Optional signing	Two 36"x12" (R6-1), two 24"x24" (R5-1), two 30"x18" (R5a-1), one 24"x24" (R3-1), and one 24"x24" (R3-2)	Low	—	Low	\$282 per site (TAPCO 2015)
	Two 54"x18" (R6-1), two 30"x30" (R5-1), two 36"x24" (R5a-1), one 30"x30" (R3-1), and one 30"x30" (R3-2)	Low	—	Low	\$452 per site (TAPCO 2015)
	Two 54"x18" (R6-1), two 36"x36" (R5-1), two 42"x30" (R5a-1), one 36"x36" (R3-1), and one 36"x36" (R3-2)	Low	—	Low	\$559 per site (TAPCO 2015)
Double minimum signing	Four 36"x12" (R6-1), four 24"x24" (R5-1), and four 30"x18" (R5a-1)	Medium	—	Low	\$404 per site (TAPCO 2015)
	Four 54"x18" (R6-1), four 30"x30" (R5-1), and four 36"x24" (R5a-1)	Medium	—	Low	\$668 per site (TAPCO 2015)
	Four 54"x18" (R6-1), four 36"x36" (R5-1), and four 42"x30" (R5a-1)	Medium	—	Low	\$789 per site (TAPCO 2015)
Double optional signing	Four 36"x12" (R6-1), four 24"x24" (R5-1), four 30"x18" (R5a-1), one 24"x24" (R3-1), and one 24"x24" (R3-2)	Medium	—	Low	\$484 per site (TAPCO 2015)

Countermeasure	Description	Estimated Effectiveness	Effect on Crash Frequency	Estimated Cost	Actual Cost
	Four 54"x18" (R6-1), four 30"x30" (R5-1), four 36"x24" (R5a-1), one 30"x30" (R3-1), and one 30"x30" (R3-2)	Medium	–	Low	\$786 per site (TAPCO 2015)
	Four 54"x18" (R6-1), four 36"x36" (R5-1), four 42"x30" (R5a-1), one 36"x36" (R3-1), and one 36"x36" (R3-2)	Medium	–	Low	\$953 per site (TAPCO 2015)
Improved signage and lighting	Improving ramp terminal conditions with oversized retroreflective signs and illuminated approaches	Medium	–	High	\$5,000 to \$15,000 per site (FHWA 2014d)
Radius at corners	Angular or tight radii make wrong-way movements difficult	Medium	–	Low	–
Raised median	Discourages wrong-way left turn entry onto interchanges: diamond, parclo, and full cloverleaf	Medium	–	Medium	–
Channelization	Devices to direct vehicles to the correct path, block, or restrict undesired movements	Medium	–	Low	–
Sight distance	Moving stop lines forward (50-60%) of the way through the intersection (WSDOT 2013)	Low	–	Low	–
ITS technologies	Video detection and TMS notification	Low	–	High	–
	Two standalone flashing LED wrong-way signs synchronized with traffic sign phase (NTTA 2009)	High	–	High	\$4,000 per site and \$450 for software (NTTA 2009)
	Pavement embedded sensors and LED warning alerts	High	–	High	–
	Inductive loops and TMS notification software (NTTA 2009)	Low	–	High	\$10,000 per site and \$55,000 for software (NTTA 2009)
	Detection, TMS notification, tracking, monitoring, driver alert, and DMS warning traffic in vicinity	High	–	Very high	–

A.1.2 Freeways

Countermeasure	Description	Estimated Effectiveness	Effect on crash frequency	Estimated Cost	Actual Cost
Avoid left side exit ramps	Drivers expect to enter freeway on the right hand side	High	–	High	–

A.1.3 Frontage Roads

Countermeasure	Description	Estimated Effectiveness	Effect on crash frequency	Estimated Cost	Actual Cost
Improved geometry and signing	Improper design of frontage roads with freeway exit ramps may cause driver confusion	High	–	Low	–

A.1.4 All Facilities

Countermeasure	Description	Estimated Effectiveness	Effect on crash frequency	Estimated Cost	Actual Cost
Alcohol ignition interlock	Driver's breath is tested by a device connected to the vehicle to detect alcohol concentration	High	–	High	\$1,200 veh/year (PennDOT 2015)
GPS vehicle alerts	The GPS provides an immediate alert to the driver when incurring on a wrong-way maneuver	High	–	Medium	\$100 to \$500 per vehicle (Garmin 2015)

A.2 Roadway Departures

A.2.1 Horizontal Curves

Countermeasure	Description	Estimated Effectiveness	Effect on crash frequency	Estimated Cost	Actual Cost
Installing reflective chevron and horizontal arrow signs	One direction road, five 18"x24" (W1-8) and one 34"x12" (W1-6)	Medium	18% (all), 25% (FI), and 35% (nighttime) (Srinivasan et al. 2009)	Low	\$184 per site (TAPCO 2015)
	One direction road, five 24"x30" (W1-8) and one 36"x18" (W1-6)	Medium	18% (all), 25% (FI), and 35% (nighttime) (Srinivasan et al. 2009)	Low	\$261 per site (TAPCO 2015)
	One direction road, five 36"x48" (W1-8) and one 48"x24" (W1-6)	Medium	18% (all), 25% (FI), and 35% (nighttime) (Srinivasan et al. 2009)	Low	\$608 per site (TAPCO 2015)
	Bidirectional road, ten 18"x24" (W1-8) and two 34"x12" (W1-6)	Medium	18% (all), 25% (FI), and 35% (nighttime) (Srinivasan et al. 2009)	Low	\$369 per site (TAPCO 2015)
	Bidirectional road, ten 24"x30" (W1-8) and two 36"x18" (W1-6)	Medium	18% (all), 25% (FI), and 35% (nighttime) (Srinivasan et al. 2009)	Low	\$522 per site (TAPCO 2015)
	Bidirectional road, ten 36"x48" (W1-8) and two 48"x24" (W1-6)	Medium	18% (all), 25% (FI), and 35% (nighttime) (Srinivasan et al. 2009)	Medium	\$1216 per site (TAPCO 2015)
Installing warning, chevrons signs, and flashing beacons	Bidirectional road, two solar flashing LED beacons, two 36"x36" (W1-1), two 24"x30" (W13-1P), and ten 36"x48" (W1-8)	High	47.6% (all), 38.2% (FI), and 76.9% (nighttime) (Montella 2009); 30% (all) (Gan et al. 2005)	High	\$4,871 per site (TAPCO 2015)
Installing dynamic flashing chevrons	Single direction solar flashing LED chevrons signs along curve	Medium	–	High	\$15,000 per site (complete system) (TAPCO 2015)

Countermeasure	Description	Estimated Effectiveness	Effect on crash frequency	Estimated Cost	Actual Cost
Installing dynamic speed warning signs	Solar flashing LED dynamic sign provided the approaching speed of vehicle prior entering the curve	Low	5% to 7% (all) (Hallmark et al. 2015)	High	\$2,795.00 to \$7,290.00 per device (TAPCO 2015)
Installing raised pavement markers	One hundred 2"x4" two sided reflection markers	Medium	Radius > 1,640 ft, 33% to -13%* (nighttime), inconclusive results (Bahar et al. 2004)	Low	\$155 per site (TAPCO 2015)
	One hundred 8"x8"x3.25" pyramid shape two sided reflection markers	Medium	Radius > 1,640 ft, 33% to -13%* (nighttime), inconclusive results (Bahar et al. 2004)	Medium	\$1,795 per site (TAPCO 2015)
	One hundred 8"x8"x3.25" pyramid shape four sided reflection markers	Medium	Radius > 1,640 ft, 33% to -13%* (nighttime), inconclusive results (Bahar et al. 2004)	Medium	\$1,995 per site (TAPCO 2015)
	One hundred solar LED 4"x4" one side illumination markers	Medium	Radius > 1,640 ft, 33% to -13%* (nighttime), inconclusive results (Bahar et al. 2004)	High	\$5,475 per site (TAPCO 2015)
	One hundred solar LED 4"x4" two side illumination markers	Medium	Radius > 1,640 ft, 33% to -13%* (nighttime), inconclusive results (Bahar et al. 2004)	High	\$6,295 per site (TAPCO 2015)
Implementing rumble strips/stripes	Centerline rumble strips on tangent sections	Low	22% to -10%* (FI rural area) (Torbic et al. 2009)	Low	\$0.10 to \$1.20 per linear foot
	Edge line in curves	Medium	15% (all) (Pitale et al. 2009)	–	(FHWA 2014b)
Installing roadside delineators	White flexible reflective delineator on both sides of the horizontal curve (30 units)	High	45% (FI) (Elvik et al. 2004)	Low	\$747 per site (TAPCO 2015)
Widening edge lines	4 to 6 and 8 inch wide (all materials)	Medium	22 to 25% (FI) (Potts et al. 2011)	Low	\$0.05 to \$1.40 per foot (FHWA 2010)
Pavement symbols, optical speed, and transverse bars	Pavement marking indicating the proximity of a horizontal curve and speed awareness	Medium	–	Low	\$0.05 to \$1.40 per foot (FHWA 2010)

Countermeasure	Description	Estimated Effectiveness	Effect on crash frequency	Estimated Cost	Actual Cost
Pavement high friction treatment	Increasing coefficient of friction of pavement to prevent lane departure, specially under severe weather conditions	Medium	25% (all), 14% (FI on wet pavement), and 25% (fatal on sharp curves) (Nambisan and Hallmark 2011, Julian and Moler 2008); 24% (all), 57% (all under wet pavement) (Harkey et al. 2008)	High	\$19 to \$35 per square yard. A project with 750 square yard surface ranges between \$14,000 to \$16,000 per site (FHWA 2014c)

* Negatives values represent increase in crashes

A.3 Intersections

A.3.1 Signalized Intersections

Countermeasure	Description	Estimated Effectiveness	Effect on crash frequency	Estimated Cost	Actual Cost
Increase clearance signal interval	Increase of all red (1.1 second)	Medium	20% (all), 14% (FI), 20% (rear end), and 3% (angle) (Srinivasan et al. 2011)	Low	–
Change left turn phase from permissive to protected-permissive	One or more approaches treated	Low	4% (FI overall intersection crashes) and 8-21% (all left turn opposing through crashes) (Srinivasan et al. 2011)	Low	–
Installation of flashing yellow	Left turn phase before treatment: Permissive or combination of permissive and permissive-protective	Medium	25% (all), 37% (all left turn crashes) (Srinivasan et al. 2011)	Low	–
	Left turn phase before treatment: Protective-permissive	Low	8% (all) and 19% (all left turn crashes) (Srinivasan et al. 2011)	Low	–
Installing dynamic warning flashers	Located upstream of intersection approaches to alert drivers of phase changing as the driver approaches to the intersection. Solar powered with one or two LED flashing beacons	Medium	18% (FI), 21% (rear-end), and 26% (angle) (Srinivasan et al. 2011)	Medium	\$1,800 to \$2,800 per device (TAPCO 2015)
Installing red light cameras	Provider service to fine red light running violators	Medium	Angle: 25% (all) and 16% (FI); rear-end: -15%* (all) and -24%* (FI) (Council et al. 2005)	Low	Self-financed programs
Improved signal visibility	Improved or replace signal sized lenses and added reflective tape to existing or new backboards	Low	7% (all), 9% (PDO), and 7% (nighttime) (Sayed et al. 2007)	Low	–
	Backplate and retroreflective edge signal head	Medium	15% (all) (Sayed et al. 2005)	–	

* Negatives values represent increase in crashes

A.3.2 Stop-Controlled Intersections

Countermeasure	Description	Estimated Effectiveness	Effect on crash frequency	Estimated Cost	Actual Cost
Installing solar LED stop signs	Improving the visibility of stop signs with LED devices and sign size	Medium	42% w/ 95% CI between 0–71% (angle) (Davis et al. 2014)	Medium	\$1,400 to \$1900 per device (TAPCO 2015)
Installing alert flashing signs	Flashing beacons in combination with stop or entering when flashing signs	Medium	25% (all) (Simpson and Troy 2013); 5% (all), 10% (FI), and 13% (angle) (Srinivasan et al. 2008)	Medium	\$1,800 to \$2,800 per device (TAPCO 2015)