Effects of Intersection Features on Wrong-Way Driving

by

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Abstract

Wrong-way driving (WWD) crashes are much more likely to result in serious injuries or fatalities compared to all other crashes. The analysis performed in this dissertation shows, specifically, that the risk of a fatality is 20 to 27 times greater for WWD crashes compared with all other types of crashes. Improperly designed intersections can lead to operational and safety concerns along roadways when vehicles attempt to enter or exit roadways. WWD entries were mostly from median-crossroad intersections on roadways, where driver view of the intersection entrance can be restricted by various intersection features. Roadways with access that are not properly managed can experience lower safety. Due to limitations in the current "3Es of Traffic Safety" strategies (Engineering, Education, and Enforcement) on WWD prevention, this dissertation suggests the need for new safety countermeasures for reducing WWD crashes on roadways. To reduce WWD crashes, the drivers must be stopped from entering the wrong way in the first place. Wrong-way entry points are where drivers started to make a wrong-way maneuver and enter the wrong side of a roadway. Thus, this dissertation focuses on the wrong-way entry points addressing the effects of intersection features on WWD through evaluating the role of all components of the intersection traffic system that may restrict driver's view and maneuvers, They are turn-prohibition signal control, signing and pavement marking traffic control devices, roadway geometric design elements, access management, intersection balance, length of median barrier, median type and width, as well as median opening treatments. It proposes a comprehensive systematic approach containing innovative safety countermeasures that are

"practice ready" to reduce WWD entries. Data collected include crash data, field data, survey data, simulation data, and Naturalistic Driving Study (NDS) data. Statistical analyses and 3D simulation are used to accomplish the objectives of this study.

Dedication

I dedicate my dissertation work to my beloved family, whose support, encouragement, and constant love surround me every day in my life. I give my deepest gratitude to them for always being there, for always supporting me, and for always letting me know their love.

I also dedicate this work to my amazing mentors and friends, who have always believed in me, always empowered me, and always let me be myself. They have been the best cheerleaders and support team to me since the day we met. My life is a better place because of those remarkable people.

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List of Abbreviations

Arizona	Department of Transportation (ADOT)
ALDOT	Alabama Department of Transportation
AASHTO	American Association of State Highway and Transportation Officials
ATSSA	American Traffic Safety Services Association
CARE	Critical Analysis Reporting Environment
CG	Circular Green
CSV	Comma-Separated Values
DNE	DO NOT ENTER
DOTs	Departments of Transportation
DUI	Driving Under the Influence
FARS	Fatality Analysis Reporting System
FHWA	Federal Highway Administration
FYA	Flashing Yellow Arrow
FCY	Flashing Circular Yellow
FCR	Flashing Circular Red
FRA	Flashing Red Arrow
GA	Green Arrow
GIS	Geographic Information System
GPS	Global Position System

HSIS	Highway Safety Information System
ITS	Intelligent Transportation Systems
LED	Light-emitting Diode
MUTCD	Manual on Uniform Traffic Control Devices
NDS	Naturalistic Driving Study
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
NTTA	North Texas Tollway Authority
OR	Odds Ratio
Parclo	Partial Cloverleaf Interchange
PPLT	protected–permissive left-turn
RID	Roadway Information Database
SHRP 2	The second Strategic Highway Research Program’s
TCDs	Traffic Control Devices
US	United States
VTTI	Virginia Tech Transportation Institute
WSDOT	Washington State Department of Transportation
WW	Wrong Way
WWD	Wrong-Way Driving
3D	Three-Dimensional

CHAPTER 1 INTRODUCTION

1.1 Background

Wrong-way driving (WWD) is a major traffic safety hazard on roadways. WWD crashes are much more likely to result in serious injuries or fatalities compared to all other crashes. The analysis performed in the dissertation shows, specifically, that WWD crashes is 20 to 27 times more likely to cause a fatality compared with all other types of crashes, which suggests the importance of investments focused on reducing WWD crashes. Limitations have been seen in the effects of the current "3Es of Traffic Safety" strategies (Engineering, Education, and Enforcement) on reducing WWD crashes. While the total number of crashes has been decreasing in recent years, the number of WWD crashes remains unchanged in the US (*NHTSA, 2017*). It suggests the need for a "safe system approach" that addresses the role of all traffic system components that may result in crashes. In the case of WWD, the main components are the roadway, vehicle and driver. Only by addressing all these components by integrated, targeted traffic safety strategies can communities hope to achieve significant reductions in WWD crashes.

By definition, WWD comprises two main elements: 1) driving movements against the main direction of traffic flow along high-speed, physically-divided highways (i.e., freeways, expressways, and interstate highways), and 2) accidentally entering a freeway system in the wrong direction from the access on and off ramps of the highways (i.e., executing a U-turn on the freeway mainline, crossing the median through an emergency turnaround, or entering from an exit ramp) (*NHTSA, 2017*). In sum, WWD entries were mostly from intersections when used as access points to highways, including interchange terminals and median-crossroad intersections on multilane divided highways, where sight distance can be restricted by driver conditions, roadway geometry, access management, or other factors.

A wrong-way incident, despite faded, damaged, or misplaced roadway signing and marking, is likely to result from a driver misreading environmental information. Especially when it is hard to identify signing and marking in low light conditions at nighttime, people rely only on environmental information to make movements (*NHTSA*, *2017*).

Intersection sight distance, the length of roadway visible to a driver, must be provided at crossroads for the safety of both through and turning traffic. Drivers using the roadway or entering and exiting a crossroad use the sight distance provided to verify that the area is clear. Providing adequate sight distance to/from a crossroad produces a safer environment because it allows drivers to perceive the entrance roadways and to adequately adjust to potential conflicts and operating impacts. Required intersection sight distances allow vehicles to enter the roadway with only a small adjustment by the traffic along the main roadway. The required crossroad (or intersection) sight distance is defined as intersection sight distance (ISD) (*AASHTO*, *2011*). ISD is used in the study for intersection design features to allow the approaching driver to anticipate and avoid potential collisions with an unobstructed view of the entire intersection.

However, the models that the current American Association of State Highway and Transportation Officials (AASHTO) Green Book (*AASHTO*, *2011*), "A Policy on Geometric Design of Highways and Streets" (hereafter referred to as the AASHTO Green Book), provide to calculate ISD are based on the assumption that both minor and major roads are assumed to be straight without any vertical or horizontal curvature. Nevertheless in reality, vertical curves and horizontal curves exist and overlap at intersections. Also, design inconsistency is a drawback of the AASHTO methodology, because drivers' responses cannot be detected until the road is built. The existing AASHTO models may underestimate or overestimate the ISD. Therefore, in this study, intersection sight distance for interchange terminals is the length where a driver on crossroads can gather more information on the entrance ramps / lanes of the roadways. Intersection sight distance for multilane divided highway is defined as the projection of drivers' sight on the profile of the entire cross section with only headlight illumination, because the median width and the grade change at an intersection restrict the sightline when the sight distance is calculated. When a driver is completing a crossing maneuver, there must be sufficient sight distance in both directions to cross the intersecting roadway and avoid approaching traffic. If the sight distance is shorter than the entire cross-section width, the segment should be treated as two separate intersections.

Past research (*Scifres et al., 1975; ADOT, 2015; and Zhou et al., 2012*) has suggested that some minor design improvements and access management techniques at intersections can mitigate WWD crashes. Nonetheless, the crash data alone cannot determine what caused the differences in the effects of intersection features on WWD, nor does it explain why some intersections are more vulnerable to WWD maneuvers than others. Using different types of datasets rather than a crash dataset alone could reveal new insights on how different intersection design features impact drive view and maneuvers that lead to WWD crashes. Understanding the detailed effects of intersection features on drive view and maneuvers will critically support the development and application of traffic safety countermeasures at intersections to deter WWD as to reduce wrong-way crashes.

1.2 Research Objectives

This dissertation work aimed to utilize various types of data to investigate the effects of intersection features on WWD. It identifies contributing factors to identify what design features could contribute to WWD. The data collected in the study include crash data, field data, survey data, simulation data, and NDS data.

The dissertation work then aimed to evaluate how exactly each design feature affect intersection sight distance. The components of the intersection traffic system that may restrict sight distance, including turn-prohibition signal control, signing and pavement marking traffic control devices, roadway geometric design elements, access management, intersection balance, length of median barrier, median type and width, as well as median opening treatments.

Results of the design feature analysis were used to develop innovative countermeasures to deter drivers from entering wrong way at intersections where WWD originated. It demonstrates "practice ready" countermeasures to address intersection sight distance issues to help drivers with decision making and maneuvers to prevent drivers from entering wrong ways in the first place. Corresponding recommendations are made on best design practice and guidelines that may discourage wrong-way entries, which, ultimately, reduce WWD crashes. An integrated approach is then used to develop, implement, and coordinate countermeasures that address each of those contributing factors.

CHAPTER 2 LITERATURE REVIEW

2.1 WWD Studies

WWD is a major traffic safety hazard on highways. By definition, WWD on highways refers to driving movements against the direction of travel required by the traffic control devices (i.e., signs, pavement markings, and signals) along high-speed, physically-divided highways (i.e., principal arterials, expressways, freeways, and interstate highways) (*NHTSA, 2017*). This section summarizes existing literatures about WWD on divided highways as well as those on freeways in the US and other countries. It provides perspectives of the problem, the latest knowledge, and potential prevention methods of WWD.

2.1.1 WWD Studies for Divided Highways

In 1973, the Virginia Highway Department and the Department of State Police performed a study to determine means for alleviating WWD on four-lane divided highways. The data were obtained from a 25-month survey of incidents of WWD on 2,000 miles of Virginia's divided highways, and investigations of the physical aspects of interchanges where wrong-way incidents occurred within the past 3 years. However, results were only given for wrong-way entries at interchanges (*Vaswani, 1977*).

In 1974, the Indiana State Highway Commission participated in a study of wrong-way movements on divided, rural highways in Indiana during the period 1970-1972 (*Scifres et al., 1975*). Field investigations were made at each crash site to supplement crash records. The study revealed 39 deaths resulted from 96 WWD crashes. Results showed that 55% (42 of 77) of drivers were physically impaired, including Driving Under the Influence (DUI), older (i.e., over 65), and fatigued; conditions at the wrong-way entry locations were darkness, low land use, or low traffic volume. Proposed measures included improving lighting, raising the elevation of crossroads, making medians more distinct, and the use of simple configurations. At certain locations, the use of additional channelization and barrier curbs could direct traffic in right ways and block wrong-way movements. The study also indicated that an estimated:

- 40 percent of drivers making WWD entries emerged from intersections with crossroads;
- 25 percent originated from business establishments, such as gas stations and motels;
- 20 percent originated from a) residential areas, crossovers, beginnings of divided sections, and construction sites, or b) associated with U-turns and median openings; and
- The origins of the remaining 15 percent were unknown.

The WWD crash analysis by the Arizona Department of Transportation (ADOT) (*2005*) indicated 91 people died in 245 wrong-way crashes in Arizona over 10 years (2004-2014). Twenty-five percent of wrong-way crashes were fatal compared to 1% of all types of crashes on divided highways. The crash analyses revealed most wrong-way crashes occurred at nighttime, on weekends, and in urban areas; 65% of wrong-way drivers were impaired; 65% of wrong-way drivers were male. To help combat WWD crashes, ADOT plans to install a wrong-way detection and warning system on divided highways that will integrate readily available technologies with existing infrastructure. The system will detect and track a wrong-way driver upon entry, inform the errant driver, notify the Traffic Operations Center and law enforcement instantly, and warn right-way drivers.

Few studies address WWD on divided highways. Previous findings on countermeasures are mostly from an engineering perspective. In general, all studies on WWD crashes on divided highways show similar trends. Table 1 summarizes past studies and the recommended countermeasures.

TABLE 1 Summary of WWD Studies on Divided Highways

State	Study Period	Countermeasures Recommended	Reference
Indiana	1970-1972	Night lighting; Raising the elevation of crossroads; Making medians more distinct; Use of simple understandable configurations; Use of additional channelization and barrier curbs.	Scifres et al., 1975
Virginia	1970-1976	Having DO NOT ENTER (DNE) and WRONG WAY (WW) signs visible in the area covered by a car's headlights and visible to the driver from the decision point on each likely wrong-way approach.	Vaswani, 1977
Arizona	2004-2014	Wrong-way detection and warning system with current ITS technology.	ADOT, 2015

This dissertation performed an in-depth investigation of WWD crashes that occurred on divided highways in Alabama from 2009 to 2013. During this period, WWD crashes on divided highways resulted in 1.29 fatalities per fatal crashes (18 fatalities in 14 fatal crashes), while this value for all crashes was 1.13 (328 fatalities in 291 fatal crashes). 12.5% of wrong-way crashes were fatal compared to 0.6% of all types of crashes. The research helped fill the gap by providing information on contributing factors and countermeasures for WWD on divided highways. It indicates the most common at-fault driver conditions are DUI, above 65 years of age, and male. Most vehicles involved were passenger cars. Air bags were deployed in most crashes, another indicator of the crash severity. The probability of a fatality or a severe injury is greater when the occupants were not wearing seatbelts. Limited visibility was also a contributing factor in WWD crashes.

The proportion of crashes of a specific type that result in a fatality, a K code using the KABCO Injury Classification Scale and Definition, is an excellent quantitative measure of the risk of a fatality due to this crash type. This data is available for the states of Alabama (AL) and Arizona (AZ). As summarized in Table 2, the proportions of WWD crashes on divided highways in AL and AZ that are fatal crashes are 12.5% and 25%, respectively. The proportions of all crashes in AL and AZ that are fatal crashes are 0.6% and 1.0%, respectively. As an excellent quantitative measure of the comparative risk of a fatality due to a crash type, the ratio of the fatalities by WWD over the fatalities by all crashes are 20.4 and 25.0 for Alabama and Arizona, respectively. This means that the risk of a fatality is 20 to 25 times greater for a WWD crash compared to all crashes. Although no data was found for other states, it is the author's opinion that the data from Alabama and Arizona may be indicative of the comparative risk in the entire country.

TABLE 2 WWD Significance Assessment by Fatality

	State	
	Alabama	**Arizona**
Crash Type	**% Fatalities**	
WWD	12.5	25
All types	0.6	1
Comparative risk: WWD fatal crashes/all fatal crashes	20.4	25.0

2.1.2 WWD Studies on Freeways

2.1.2.1 Contributing Factors

Previous studies concluded that driving under the influence (DUI), driver age, driver gender, driving fatigue, darkness, weekends, and urban areas were the primary factors of wrong-way crashes. Road design configuration was not an identified factor, but some indicated that the reason for WWD "does not lie in the driver as such but in inadequate road surfaces at specific spots as well as in traffic signalization systems" (*Kemel, 2015*). Others speculated that insufficient signage and pavement marking at a median-crossroad intersection could lead to wrong-way crashes.

2.1.2.2 Countermeasures

Countermeasures strive to provide positive warnings to drivers at the earliest decision points and/or supplemental warning after they begin traveling in the wrong direction. The common countermeasures for WWD include engineering (signage, pavement marking, roadway geometry, and intelligent transportation systems (ITS), enforcement (emergency response, confinement, and radio messages), and education (training) (*Cooner et al., 2004; Schrock et al., 2006; and Pour-Rouholamin, 2014*). They differ on feasibility, applicability, effectiveness, implementation priority, and associated cost. A combination of them may reduce wrong-way crashes. See Table 3 for applicable engineering and enforcement countermeasures to WWD on divided highways.

TABLE 3 Summary of WWD Countermeasures on Engineering (a) and Enforcement (b) in the US and Other Countries

(a)

State	**Study Period**	**Countermeasures Recommended**	**Roadway Type**	**Reference**
Indiana	1970-1972	Night lighting; Raising the elevation of crossroads; Making medians more distinct; Use of simple understandable configurations; Use of additional channelization and barrier curbs.	Divided Highway	Scifres and Loutzenheiser, 1975
Virginia	1970-1976	Having DO NOT ENTER (DNE) and WRONG WAY (WW) signs visible in the area covered by a car's headlights and visible to the driver from the decision point on each likely wrong-way approach.	Divided Highway and Freeway	Vaswani, 1977
California	1983-1987	Continuing monitoring of WWD incidents; Periodic review of fields; Use of detectors and cameras; Pavement lights.	Freeway	Copelan, 1989
New	1990-2004	Improved lighting and signage at	Freeway	Lathrop et al.,

Mexico		possible WWD entry points with crash records.		2010
North Carolina	2000-2005	Embedded sensors; Video detection systems and flashing lights; Spikes and other barriers; Sensor video information for making modifications.	Freeway	Braam, 2006
Texas	1997-2000	Traditional signage and pavement marking (DNE and WW signs, WW arrows, etc.); Innovative signage (supplemental placards, red retroreflective strips on signs supports, etc.); Advanced technologies (detectors and warning signs).	Freeway	Cooner et al., 2004a; Cooner et al., 2004b
Illinois	2004-2009	Red retroreflective tape on sign supports; Lane-line extensions for complex intersections.	Freeway	Zhou et al., 2012
Michigan	2005-2009	Red retroreflective strips on sign supports; WW pavement arrow; lane-line extensions; Red delineators on marking.	Freeway	Morena and Leix, 2012
Arizona	2004-2014	Wrong-way detection and warning system with current ITS technology.	Divided Highway	ADOT, 2015
Switzerland	2003-2005	Radio warning messages; Directional arrows at one-way roads; Double-sided No Entry signs.	Freeway	Scaramuzza and Cavegn, 2007
France	2008-2012	Sign-based countermeasures; Changes in the infrastructure geometry; Monitoring divided roads.	Freeway	Kemel, 2015
Netherlands	1996-1998	Placing extra arrows on the road surface to show the correct direction; Additional “Go Back” sign installed at the bottom of the DNE sign.	Freeway	SWOV, 2009
Finland	1999-2002	Appropriate use of guidance and signing traffic control devices (TCDs).	Freeway	Karhunen, 2003
Japan	2005-2009	Appropriate guide signs; ITS applications such as onboard navigation systems and advanced sensing technologies; Road-vehicle communication technologies; Pavement arrow markings.	Freeway	Xing, 2015

(b)

Agency	**Enforcement Recommended**	**Note**	**Roadway Type**	**Reference**
New Mexico	Intervention strategies to reduce the prevalence of impaired driving.	Not Available	Freeway	Lathrop et al., 2010
French Motorway Company	Implementing wrong-way driver confinement as an emergency management strategy to activate “closure of toll barriers, tunnels, and motorway access” in the direction of the zone concerned; broadcasting alert over a radio frequency following the detection of the wrong-way	Does not protect vehicles already driving inside the confined area.	Freeway	Vicedo, 2006

	incident.			
Caltrans	Parking lot spike barriers; Wrong-way crash reports extraction from the crash database every year and generated wrong-way crash concentrations; Field investigations at the locations where wrong-way crashes were most prevalent and where possible wrong-way entries had occurred; Reports documented where wrong-way crashes were concentrated, with descriptions of observed deficiencies; A checklist for field inspections to check the conditions of traffic signs and pavement markings periodically.	According to the 1989 Caltrans survey, no states had developed special devices to physically prevent wrong-way entries and no traffic engineers endorsed the use of parking lot spike barriers or raising curbs.	Freeway	Caltrans, 2004
Georgia Department of Transportation (GDOT)	Raised physical, curb-like barriers to impede the wrong-way driver; Enhanced legislation and DUI checkpoint programs.	This device was not feasible for reasons similar to those for the directional in-pavement spike.	Freeway	Vaswani, 1977
Texas Department of Transportation (TxDOT)	Reviewing wrong-way entry issues and suspected problem locations; A checklist for field inspections to check the conditions of traffic signs and pavement markings periodically.	Effective in pinpointing deficiencies in the field.	Freeway	Cooner et al., 2004a; Cooner et al., 2004b
TTI	A wrong-way entry checklist for reviewing wrong-way entry issues and suspected problem locations; Coordinating with the primary 911 public safety answering points to share information on reports of wrong-way movements.	None.	Freeway	Finley et al., 2014.
North Texas Tollway Authority (NTTA)	Enhanced legislation and DUI checkpoint programs; Deployment of a wrong-way detection and warning system.	Successful only if there is sufficient staff to receive the alert and quickly respond to the wrong-way incident.	Freeway	NTTA, 2009

2.1.2.3 Summary

Findings from the past studies for divided highways and freeways were summarized. WWD entries were mostly from median-crossroad intersections on divided highways and interchange terminals on freeways, where sight distance can be restricted by driver conditions, darkness, roadway geometry, or other factors. Countermeasures were very limited and from engineering judgments without data support. Insights on contributing factors and countermeasures from freeway WWD studies that apply to divided highways were given.

2.2 Wrong-Way Entry Points

One of the most challenging aspects of studying wrong-way crashes is identifying wrong-way entry points where drivers first turned the wrong direction on roadways. Based on previous studies, the most common WWD entries on divided highways occur when drivers (a) miss an intended exit, (b) choose the nearby wrong way instead of the divided right way on the far side when joining from a non-freeway crossroad, (c) enter a roadway going the wrong direction at the road's terminal, (d) make a U-turn and misunderstand that the next lane will be in the opposite direction, or (e) attempt to get back on the main road after stopping at a service or parking area (*Scifres et al., 1975; Vaswani, 1977; ADOT, 2015; Finley et al., 2014; and NTTA, 2009*).

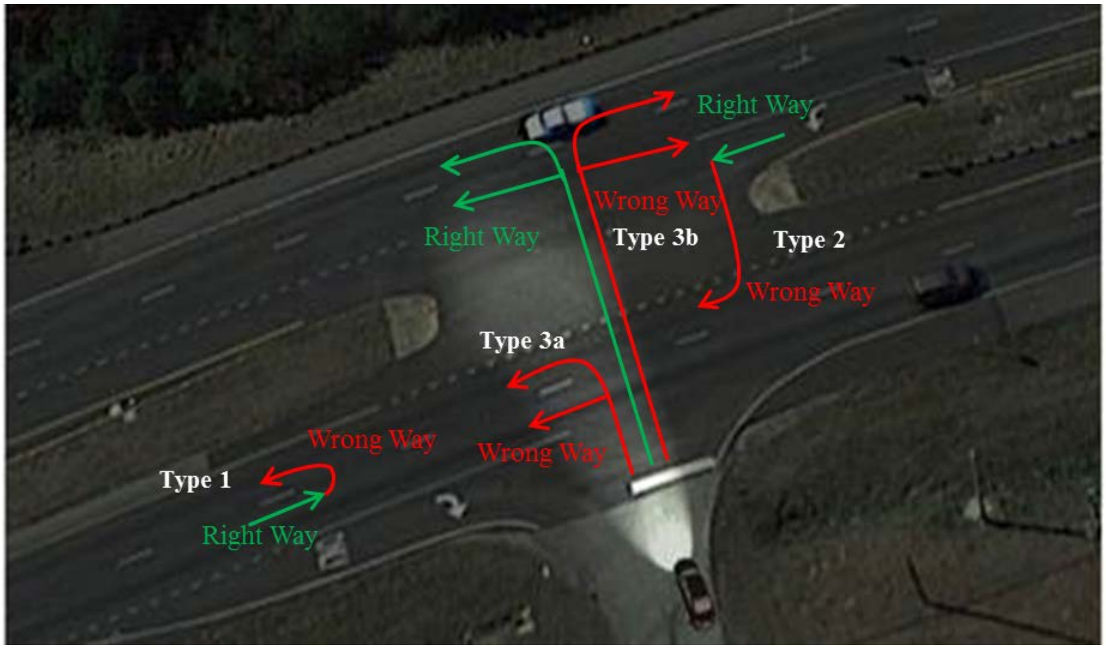
Several previous studies used information sources such as police crash reports, surveys, and images from camera surveillance systems to determine where wrong-way movements originated. Most entry points for two-thirds of the crashes were unknown because wrong-way drivers usually could not provide information due to impaired condition or injury/death in crashes.

Scifres et al. (*1975*) showed that on non-interstate four-lane divided highways, 40% of drivers making WWD entries emerged from intersections with crossroads; 25% originated from business establishments such as gas stations and motels; and 20% originated from residential areas, crossovers, beginnings of divided sections, and construction sites, or were associated with U-turns and median openings. The origins of the remaining 15% were unknown.

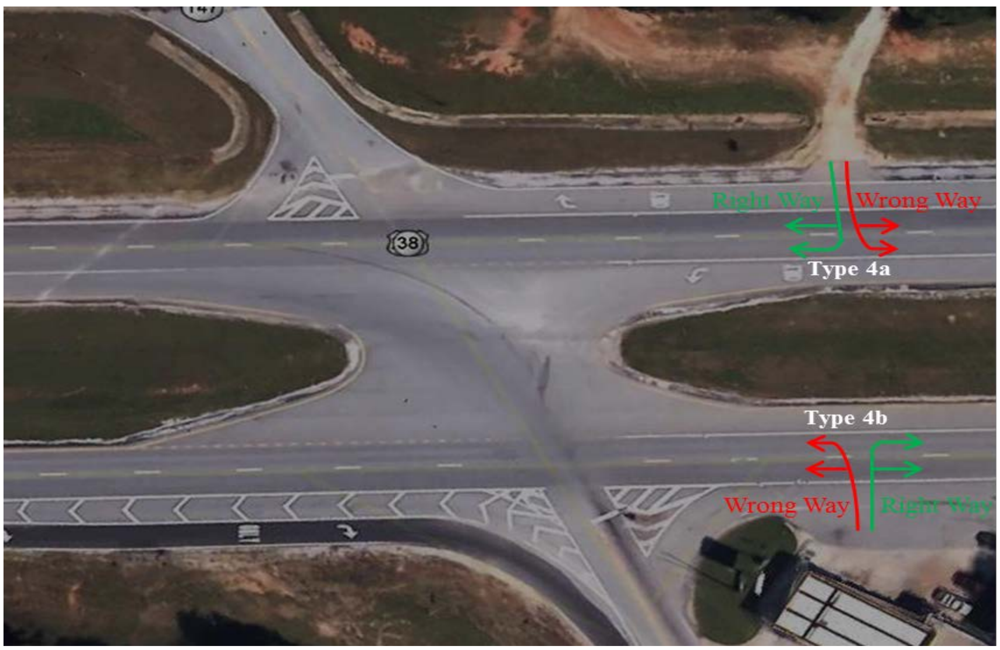
According to the research results, WWD on divided highways without fully controlled access are categorized into four types. The first three types are shown in Figure 1(a) and the fourth type is shown in Figure 1(b). They are: Type 1, U-turns; Type 2, crossing the median; Type 3a and 3b, illegal access from access points with a median opening, which involves two situations: a left turn into the near lanes, or a right turn into the lanes on the far side of the median); and Type 4a and 4b, left-turns at access points with no median opening: by either intent or mistake, drivers turn left at a minor access point and travel the wrong way, when they should have turned right, travel to the next median opening for a U-turn to proceed in the desired direction. See Table 4 for more details.

TABLE 4 Wrong-Way Movements on Divided Highways

Type No.	Short name	Narrative Description (shown in Figures 1 (a) and 1 (b)
		The descriptions in this column are based on north being at the top of Figures 1 (a) and 1 (b).
1	Illegal U-turn	A vehicle traveling eastbound (EB) makes an illegal U-turn and then proceeds westbound (WB) using the EB lanes, traveling the wrong way.
2	Illegal Median Crossing	A vehicle on the WB approach makes a left turn and then proceeds WB using the EB lanes, traveling the wrong way.
3a	Illegal Access with a median opening	A vehicle on the northbound (NB) approach makes a left turn and then proceeds WB using the EB lanes, traveling the wrong way.
3b	Illegal Access with a median opening	A vehicle on the NB approach makes a right turn and then proceeds EB using the WB lanes, traveling the wrong way.
4a	Illegal Access without a median opening	A vehicle on a southbound (SB) driveway on the WB lanes makes a left turn and then proceeds EB using the WB lanes, traveling the wrong way.
4b	Illegal Access without a median opening	A vehicle on a NB driveway on the EB lanes makes a left turn and then proceeds WB using the EB lanes, traveling the wrong way.



(a)



(b)

FIGURE 1 Drivers' wrong-way movements on divided highways

2.3 Signal Control for Turn-Prohibition at Wrong-Way Entry Points

This section summarizes existing practices and past studies for steady green signal control for turn-prohibition at wrong-way entry points.

2.3.1 Current Standards

For decades, traffic engineers have relied on the MUTCD for installations of traffic control devices. Regarding intersection control, it addresses the design and application of traffic signals and traffic signs, used in combination or individually.

In the 2009 MUTCD Section 4D.04 (*2009*), the rules of CGs are “Vehicular traffic facing a Circular Green signal indication is permitted to proceed straight through or turn right or left or make a U-turn movement except as such movement is modified by lane-use signs, turn-prohibition signs, lane markings, roadway design, separate turn signal indications, or other traffic control devices.” For GAs, “Vehicular traffic facing a Green Arrow signal indication that displayed alone or in combination with another signal indication is permitted to cautiously enter the intersection only to make the movement indicated by such arrow, or such other movement as permitted by other signal indications displayed at the same time.” This clarifies that drivers can proceed only in the direction of the arrow (assuming no other green signal) through the junction.

A recommendation in 2009 MUTCD Section 4D.05 (*2009*) states: “If not otherwise prohibited, a steady straight-through Green Arrow signal indication may be used instead of a circular green signal indication in a signal face on an approach intersecting a one-way street to discourage wrong-way turns.” Accordingly, GAs are often seen on one-way streets in urban areas. They can also be supplemented by regular or LED turn-prohibition signs.

For supplemental signs, MUTCD states “Installed traffic signs must comply with the MUTCD’s mandatory, advisory, and permissive requirements: Turn-prohibition signs combined with a Circular Green or Green Arrow warn road users of required, permitted, and prohibited traffic movements;" and "Both No Right/Left Turn signs can be placed with traffic control signals (adjacent to a signal face for a better view by road users)". Different sizes (inches) of Turn-prohibition signs, i.e. 24X24, 36X36, 48X48, and other, various signal indication and sign arrangements are allowed (*FHWA, 2009*). See Table 5 for examples.

TABLE 5 Turn Prohibition Signs (*FHWA, 2009*)

Sign	Size (inch)
 or 	24X24 36X36 48X48 Other
	36X12 54X18 Other
	18X24 24X30 30X36 36X48 48X60 Other

The Uniform Vehicle Code (*NCUTLO, 2000*) states, "Vehicular traffic facing a circular green signal may proceed straight through or turn right or left unless a sign at such place prohibits either such turn," and "Vehicular traffic facing a green arrow signal, shown alone or in combination with another indication, may cautiously enter the intersection only to make the movement indicated by such arrow, or such other movement as is permitted by other indications shown at the same time". This statement is consistent with guidelines in MUTCD (*FHWA, 2009*).

2.3.2 Previous Studies

Past studies addressed the effects of certain types signal indications, mostly about the flashing yellow arrow (FYA), flashing circular yellow (FCY), flashing circular red (FCR), and flashing red arrow (FRA), but few analyzed the effectiveness of CGs and GAs or compared the differences between them.

Brehmer et al. (*2003*) compared the protected–permissive left-turn (PPLT) phasing schemes to identify the “best” exclusive left-turn signalizations. It surveyed current practices, analyzed crash and operational data, and studied drivers’ understanding using a full-scale driving simulator. Conclusively, drivers paid more attention and better understood the FYA than other displays, including CGs and GAs, but no comparison was made between the green signals.

Noyce et al. (*2000*) examined traffic operations in the field and quantified the capacity and delay effect of various protected–permissive indications, including CG, FCY, FCR, and FRA. The results of the study found that there were no significant differences in the saturation flow rates or start-up lost time for the four indications analyzed. The start-up lost time and response time were found to be significantly influenced by the PPLT signal phasing and signal arrangement, but not by the signal display. The follow-up headway time was the shortest for CG, followed in increasing length by FCY, FCR, and FRA.

Noyce et al. (*2007*) conducted crash analyses to evaluate the safety effectiveness of the FYA permissive left-turn indication. Data collected included crashes, signal phasing, vehicle flow rate, posted speed limit, and intersection geometry at 120 locations in the US. Results showed safety improvements were observed at the study intersections operating with PPLT control before the implementation of FYA with PPLT phasing.

Noyce and Kacir (*2001*) evaluated safety, operational performance, and driver understanding of CG, FYA, FCY, FRA, and FCR through a survey distributed throughout the US. Results indicated a significantly higher correct response rate to the scenarios with the flashing red and yellow indications than to those with the CG indication. Higher correct response rates at the flashing permissive indications were also observed for demographic groups such as the elderly, inexperienced drivers, and drivers with limited education.

Knodler et al. (*2007*) conducted a driver comprehension survey of the solid yellow arrow indication used with the FYA. The objective was to determine whether driver understanding of the solid yellow arrow changed after drivers were introduced to the FYA. A static computer survey was used to evaluate the driver comprehension of participants in Massachusetts and Wisconsin. The research concluded that there was no evidence that the FYA indication negatively affected drivers' understanding of the solid yellow arrow.

Knodler et al. (*2005*) conducted a driver simulator study to evaluate the FYA displays with simultaneous permissive indications. The FYA used in a five-section cluster display was compared with the standard CG five-section cluster and FYA as an exclusive vertical four-section arrangement. Results showed the FYA used in a five-section cluster display had no significant impact on driver comprehension of the FYA for PPLT control.

Knodler et al. (*2006*) also evaluated the potential application of the FYA and FRA indications for PPLT control in wide medians with separated left-turn lanes. They found that significantly more drivers yielded at the intersections with the FYA display than at those with the FRA display. The FRA indication was incorrectly comprehended by a large number of drivers as a yield instead of stop movement. They suggested that FYA indications to be utilized for wide median intersections for permissive movements, but only after the FYA had been more widely implemented or accompanied by supplemental signage.

Noyce and Smith (*2003*) evaluated driver understanding of five different types of indications for PPLT control: CG, FCY, FYA, FCR, and FRA by driving simulation. The results revealed significantly higher driver understanding of the CG, FYA, and FCY than of the FRA and FCR. Flashing red indications had the most common incorrect but safe responses; more drivers proceeded through the intersection with no visible stop or yield for CGs compared with the others.

Knodler et al. (*2006*) conducted a study to analyze driver and pedestrian comprehension of the requirements for the FYA indication for PPLT applications. It showed there were significantly more “yield” and “stop-and-wait” responses at T-intersections with the FYA and a statistically higher proportion of “go” responses at T-intersections with the CG. In the absence of a pedestrian crossing signal, pedestrians were more likely to identify the correct crossing opportunities when the CG was used.

Knodler et al. (*2007*) evaluated the impact gradual implementation of the FYA for permissive indication had on drivers’ comprehension of the CG indication. They found that there was a significant increase in the number of correct driver responses to the FYA when the driver had prior education. Conclusions were that drivers were not likely to misinterpret the CG indication as “go” with FYA presence and that driver comprehension of the CG before exposure to the FYA was significantly equivalent to comprehension after exposure to the FYA.

Rietgraf et al. (*2013*) assessed differences in operation and driver behavior for three signal displays for the permissive interval in a protected–permissive left-turn phase. A comparative two-phased study was conducted based on operational data by field observations and video recordings. Phase 1 examined driver behavior at six intersection approaches in Peoria, Illinois, and showed that the FYA produced the highest proportion of safe and efficient driver actions compared with CGs and Flashing Red Arrows. In Phase 2, a comparative analysis of four intersection approaches was conducted on driver understanding; no difference was found.

Hubacher et al. (*2003*) predicted traffic accidents at CG and GA signals using configuration-specific features by Poisson regression. Configuration-specific features were surveyed (e.g., traffic lanes, road signs, and traffic lights). Traffic and accidents were recorded over five years. The study found that approach road topography was the major contributor to a GA accident: If the approach road slopes downwards, the risk of a GA accident was approximately five and a half times greater than on a level or upward sloping approach road. Obstructed vision played the major role in CG accidents: Where vision can be obstructed, the accident risk was eight times greater than where there was no comparable obstruction to vision. Other factors, like driver demographics (e.g., age, gender, etc.) and accident environment (e.g., lighting, road conditions, etc.), were more likely to contribute to accidents than traffic lights.

Recently, a freeway WWD study by Zhou et al. (*2012*) in Illinois suggested that GAs at interchange terminals may provide a better indication of allowed movements and reduce WWD, but no verification was done for the statement.

2.4 Roadway Geometric Elements at Wrong-Way Entry Points

Of interest is the application of geometric elements to deter WWD at the intersections of interchange ramps and crossroads. Past studies about use of geometric elements to reduce wrong-way entries at interchange terminals are summarized.

Eyler (*2005*) recommended using an appropriate angle for sweeping connections of exit ramps to crossroads, such as outer connections, loops, and some diamond ramps, to make interchange terminals less susceptible to WWD. For example, if left turns from exit ramps are prohibited because of a connecting one-way roadway or the presence of a raised median on the roadway, an acute angle should be used to connect exit ramps to the roadway. This is primarily due to the inherent capability of the formed acute angles with the crossroads, which causes turning movements in either direction difficult.

Cooner and Ranft (*2008*) evaluated the most effective traditional and innovative countermeasures throughout the United States to reduce wrong-way movements. They developed a typical wrong-way crash profile utilizing four years of wrong-way crash data on Texas freeways. They recommended guidelines for parclo interchange terminals. By adding raised channelizing islands separating adjacent entrance and exit ramps to reduce ramp-throat width, this geometric design can discourage wrong-way entries by making ramps uninviting to drivers, especially at multilane exit ramps.

Chassande-Mottin and Ganneau (*2008*) proposed to reduce the complexity of intersections connecting to interchanges for WWD prevention. Specifically, the use roundabouts instead of multiple islands can reduce drivers' confusion at interchange terminals as well as for control access, thereby, reducing wrong-way entries.

Zhou et al. (*2012*) conducted a WWD study to identify contributing factors and countermeasures to reduce WWD crashes on Illinois freeways. A series of geometric elements were proposed to be potential countermeasures to decrease WWD crashes in the vicinity of freeway interchanges, such as using raised median and channelizing islands, increasing the median width between exit ramps and entrance ramps, and reducing the turning radius to parclo interchanges.

The North Texas Tollway Authority (NTTA) (*2009*) investigated the effects of median modifications to reduce WWD incidents at a study location in Dallas, Texas. The location was identified to be the originating point of several WWD incidents due to a side street closely placed to an exit ramp which resulted in left-turning drivers incorrectly entering the exit ramp instead of the side street from the crossroad. To mitigate wrong-way movements, the NTTA closed the median opening to prohibit left turns into the side street. Afterwards, no WWD incidents were recorded at this location.

According to Morena and Ault (*2012*), longitudinal channelization devices can also be used as low-cost geometric countermeasures by transportation agencies to deter WWD crashes. The Michigan Department of Transportation (MDOT) applied this countermeasure to one high WWD crash location in 2010. Since 2012, no WWD crashes were reported at this location after the treatment.

No specific guideline exists for stop line positioning for left turns onto entrance ramps at parclo interchange terminals in the Manual on Uniform Traffic Control Devices (MUTCD) or AASHTO Green Book (*FHWA, 2009; and AASHTO, 2011*). In 2013, the Washington State Department of Transportation (WSDOT) (*2013*) proposed that the distance between the stop line for left turns from a crossroad and the middle of the median separating on and off ramps be no more than 60% of the entire intersection width (Figure 2). The green line illustrates the right-way movements, while the red line represents the wrong-way movements.

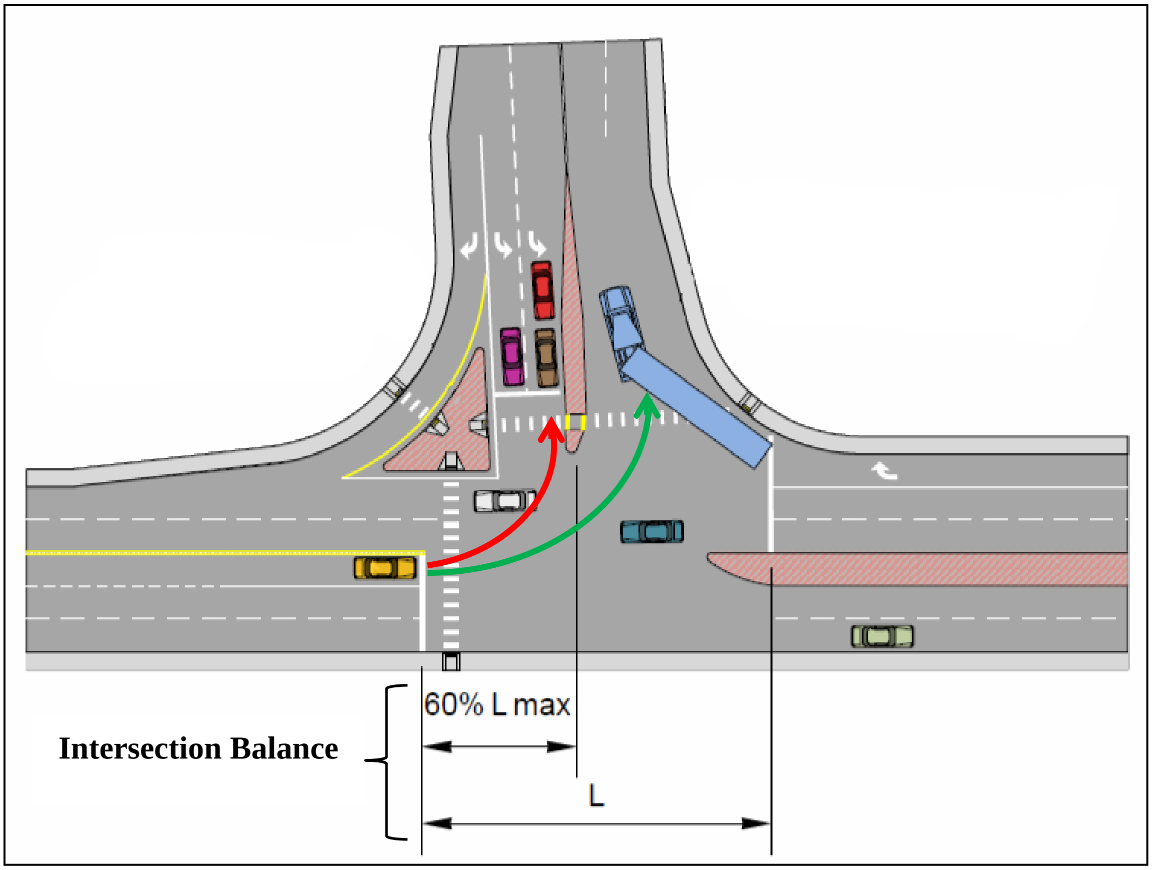


FIGURE 2. Current best practice of Intersection Balance (*WSDOT, 2013*)

However, there is lack of scientific research to support the proposals outlined by WSDOT. Figure 3 shows an example of unbalanced intersections where WWD crashes and incidents were recorded (*Morena and Leix, 2012*). The distance between the stop line for north bound left turns and the median centerline is more than 60% of the entire intersection, which makes the entrance ramp difficult to be seen by left turn drivers and creates a large turning radius.

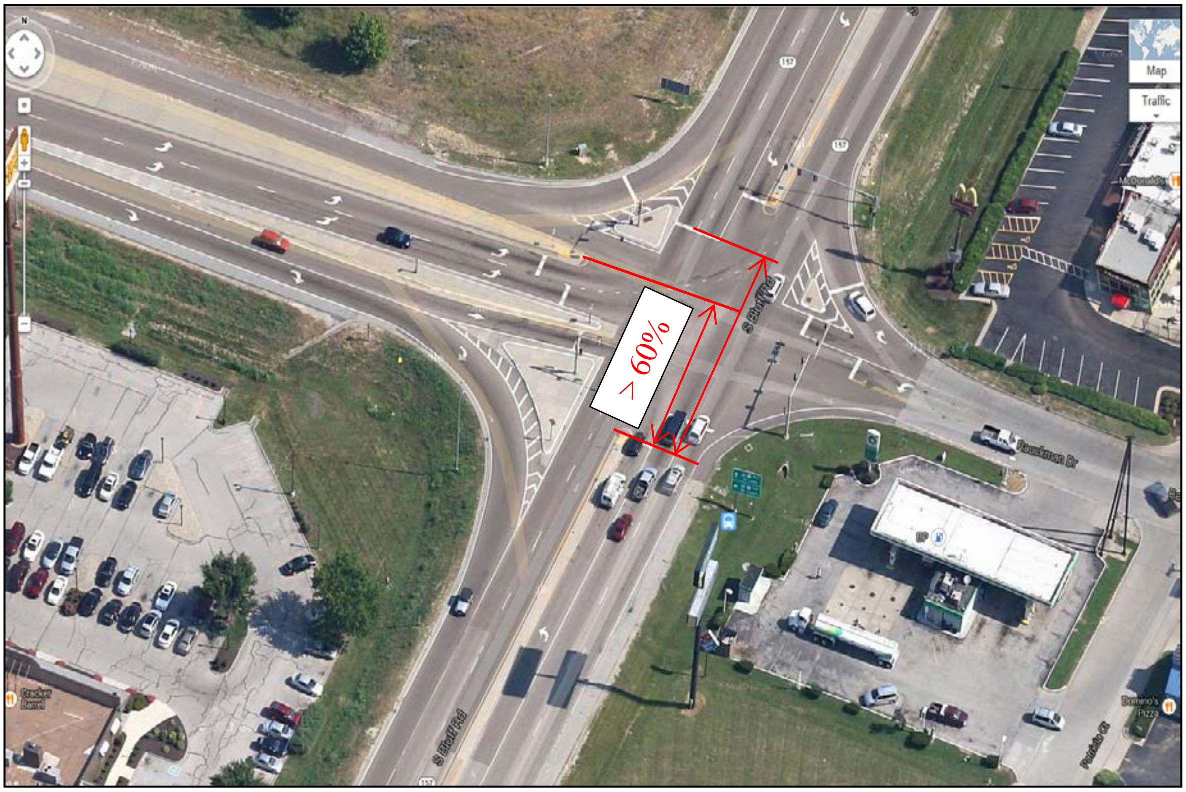


FIGURE 3 Example of unbalanced intersections

Past research proved the effectiveness of the application of some geometric elements in mitigating wrong-way movements at parclo interchange terminals, including using raised median and channelizing islands, increasing the median width between exit ramps and entrance ramps, and reducing the turning radius, but their detailed impacts on WWD movements have not been evaluated. Accordingly, statistical analyses was performed to examine how different geometric elements may factor in WWD, and then outline how the research may fill the gaps in research and address the impact of specific geometric elements to improve safety.

2.5 Intersection Balance at Wrong-Way Entry Points

This section reviews existing practices and studies on intersection layout, especially left-turn stop line positioning, three-dimensional (3D) views by left-turn drivers, and the impact of median barrier design on drivers' view of entrance ramps.

2.5.1 Stop line positioning

The MUTCD (*FHWA 2009*) indicates that "*if used, stop lines shall consist of solid white lines extending across approach lanes to indicate the point at which the stop is intended or required to be made. Stop lines should be used to indicate the point behind which vehicles are required to stop, in compliance with a STOP (R1-1) sign, traffic control signal, or some other traffic control device. If used, stop and yield lines should be placed a minimum of 4 feet in advance of the nearest crosswalk line at controlled intersections, except for yield lines at roundabouts as provided for in Section 3C.04 and at midblock crosswalks. In the absence of a marked crosswalk, the stop line or yield line should be placed at the desired stopping or yielding point, but should not be placed more than 30 feet or less than 4 feet from the nearest edge of the intersecting traveled way.*" There is no specific guideline on stop line positioning for left turns onto entrance ramps at parclo interchange terminals in the MUTCD. In 2013, WSDOT (*2013*) proposed to locate the stop line for left turns from crossroads at no more than 60% of the entire intersection width. However, no scientific research results were found to support this proposal.

2.5.2 3D view of entrance ramp by left-turn drivers

For intersection sight distance (ISD) calculation, the traditional two-stage highway design process provides separate processing of horizontal and vertical alignments. The models in the current AASHTO Green Book provide equations and charts to calculate the ISD along the intersecting highway to allow the approaching driver to anticipate and avoid potential collisions with an unobstructed view of the entire intersection. However, these models are based on the assumption that both minor and major roads are assumed to be straight without any vertical or horizontal curvature, but in reality, vertical curves and horizontal curves exist and overlap at intersections, where the AASHTO models does not apply. Also, design inconsistency is a drawback of the AASHTO methodology, because drivers' responses cannot be detected until the road is built. Researchers have suggested that the existing AASHTO models may underestimate or overestimate the available sight distance (*Easa and He, 2006; Yan and Radwan, 2008; Hassan et al., 1996; Jha et al,. 2011; Tsai et al., 2009; and Easa et al., 2004*).

Sarhan et al. (*2008*) used a reliability analysis to estimate the probability of hazard resulting from insufficient sight distance to replace the current deterministic highway design practice. The available sight distance was checked against the required stopping sight distance on an assumed road segment consisting of a horizontal curve overlapping with flat grade, crest curves, and sag curves in a cut section where the side slope restricts the sightline. Variation of the design parameters was addressed with Monte Carlo simulation. The analysis showed that calculating stopping sight distance by the current deterministic approach was not sufficient.

To remedy this, Jha et al. (*2011*) proposed the concept of 3D design to eliminate the need for post-checking to ensure design consistency and the availability of adequate sight distance. The 3D model combined horizontal and vertical alignments based on the curved parametric elements. The elements presented were rectangular and triangular. These elements were used to represent various features of the highway surface and sight obstructions, including the grade, horizontal curve, vertical curve, traveled lane, shoulder, side slope, cross slope, superelevation, lateral obstruction, and overpass. The available sight distance was calculated analytically by examining the intersection between the sight line and the elements representing the highway surface and sight obstructions. With the proposed 3D design methodology, planners and designers can simulate the entire design process at the onset.

Manoj et al. (*2009*) used a method by which the sight distance was measured using the 3D road surface, a solid cone, and a rectangular plane. The 3D cone, with its vertex at height 'h' from the road surface and line of height parallel to the tangent of the road centerline, was moved along the roadway at regular intervals. The intersection of the road surface with the cone was used to obtain the intersected surface. The variation of the tangents along the intersected surface was used to obtain the profile of the intersected road surface centerline. The variable rectangular plane was used over the intersected road surface to calculate the sight distance.

2.5.3 Length of Median Barrier on Two-way Ramps

Median barriers, such as guardrails, are longitudinal barriers most commonly used to protect motorists traveling in opposing lanes and to separate opposing directions of traffic on a divided highway. While these systems may not reduce the frequency of crashes due to roadway departure, they can help prevent a median crash from becoming a median crossover head-on collision. On the freeway system, an average of 250 people are killed annually in crossover crashes, a number which may seem small but these crashes are in fact three times more severe than other highway crashes (*FHWA 2014*).

Recommended in the AASHTO guidelines (*AASHTO, 2011*), guardrail run-out distances and length-of-need requirements depend on traffic conditions, guardrail layout, and the characteristics of the hazard to be shielded. They are independent of the terminal type, assuming that the point at which the length-of-need begins is the same for each terminal (normally 12.5 feet from the terminal's beginning). When two-way ramps are close to each other at an interchange terminal, the guardrail is installed to provide the proper length of need to protect vehicles from roadside safety hazards based on current policies (*AASHTO, 1977; and Glennon, 2002*).

However, Glennon (*2002*) found that AASHTO procedures used to find length-of-need may be flawed and determined overlong lengths of need, and many highway agencies have rejected these procedures and resorted to a wide variety of rationales for determining shorter guardrail lengths. Many guardrails installed have been so long that they actually increased the expected severity of many impacts that would otherwise encounter a relatively safe contiguous roadside near the end of the guardrail.

Morena et al. (*2014*) revealed that a guardrail installed between two adjacent exit and entrance ramps will block left-turn drivers' view of the entrance ramp terminal and increase the possibility of making a wrong left turn onto the exit ramp. The study found 10 recorded WWD crashes occurred during 2005-2009 at the interchange of I-94 at Gratiot Avenue in Detroit that were resulted from a median guardrail that extends nearly entirely to the curb lane on ramps and leaves drivers no clear view of the entrance ramp. The study recommended that median barriers on two-way ramps should not be extended all the way to the stop line.

In summary, few studies have been conducted on appropriate stop line positions or validated the practice of locating stop lines at no more than 60% of the way through the intersection to provide appropriate intersection balance at signalized ramp terminals of parclo interchanges. The concept of 3D roadway visualization may help in investigating the effects of different stop line positioning at parclo interchange terminals on drivers' perception of entrance ramp information. In addition, no guidelines were found for appropriate median barrier lengths for two-way ramps. This section was intended to validate the practice with WWD crash data analysis and drivers' sight distance calculation with a new 3D simulation methodology. The effects of median barrier length on drivers' sight distance were evaluated and proper lengths were determined for median barriers in 3D models. Corresponding recommendations were developed to improve sight distance at signalized ramp terminals of parclo interchanges.

2.6 Effects of Median Widths at Wrong-Way Entry Points

The current breakpoint (30 feet) for median widths was first brought into the 1978 Edition of the MUTCD. It was previously identified in the 1944 Uniform Vehicle Code and remained unchanged in the following versions (*NCUTLO, 1944; and FHWA, 1978*). When it is narrower than 30 feet, the median can be treated as one intersection with no extra TCDs required; otherwise it is treated as two intersections. According to Harwood et al. (*1995*), this measure was from experience, not scientific research. A recent study (*Hadi et al., 2014*) recommended that the 30 feet criterion be fully removed from the MUTCD and related figures be revised to show the modified measurement of median widths through analytical assessment of interactions of opposing left turns in the median.

There has always been a tradeoff between safety and capacity when considering median widths. While median widths are kept to a minimum to accommodate right-of-way and higher capacities on new highways, having wider medians can improve safety by providing errant vehicles with sufficient recovery area. According to Stamatiadis et al. (*2009*), various trends can be observed by changing median widths. Wider medians can help alleviate the problem of cross-median crashes, but other median-related crashes (e.g., rollover) increase with a peak at around 30 feet and a reduction afterwards.

A study by the FHWA (*Knuiman et al., 1993a*) was performed in Illinois and Utah using log-linear regression models and a negative-binomial variance function. It found a steady reduction in the accident rate with increasing median widths; this decline was more obvious for the first 30 feet of median width than after. This safety benefit was visible up to a width of 60 to 80 feet. Eighty feet is the highest value for median width range identified by the AASHTO Green Book (*2011*), within which most medians fall. Several studies have indicated that few safety benefits were seen with medians wider than 60 feet (*Knuiman, 1993b; IDOT, 2009; Lu et al., 2006; and Donnell et al., 2002*).

Harley et al. (*2008*) employed negative binomial regression models to calculate several crash modification factors associated with median widths based on ten years' data (1993 to 2002) in California. Traffic volumes, shoulder widths, and median widths were used as independent variables to predict response variables, total crashes and cross-median crashes. Median widths were found to have more considerable effects on cross-median crashes than total crashes. The effects were more evident in rural areas than in urban areas. However, no specific median width was identified as the balancing point between safety and capacity.

Khazraee et al. (*2014*) attempted to identify the appropriate minimum median width for intersection signing purposes to eventually change the definitions in current manuals and enhance safety. The minimum turning path for design passenger cars was used to assess the current definitions of minimum median width for signing purposes. It was concluded that the median width of 30 feet, suggested by the MUTCD (*10*), does not satisfy the minimum requirements for operating as two separate intersections.

Harwood et al. (*2000*) surveyed median widths at intersections of divided highways without full control of access. Accordingly, out of 43 participating states, 20 observed median-related operational problems at intersections with wide medians. Increasing potential of WWD was recognized. However, this survey did not specify any critical median width to prevent WWD.

In summary, few studies have been conducted on the impacts of varying median widths on wrong-way incidents for appropriate TCD applications. This section was to evaluate the effects of median widths to deter WWD on multilane divided highways. Results can be used to identify the proper median opening treatment and examine if the existing TCDs are sufficient to deter WWD. Recommendations can be developed for possible revisions in the MUTCD.

2.7 Naturalistic Driving Study (NDS) Data

Naturalistic Driving Study (NDS) data (including crashes, near-crashes, critical incidents, traffic conflicts, as well as ordinary driving data) enables further finer-grain analysis of the effects of roadway design elements on driver behavior (*2013*). The naturalistic driving method offers exposure information, including the frequency of behaviors in normal driving, as well as the larger context of contributing factors. The larger context for exposure enables risk estimates for various driver behaviors and for other contributing factors. Thus, using NDS data allows for empirical support to develop new guidelines and safety countermeasures to reduce crashes. With NDS data, additional analyses can be conducted to evaluate the effects of different roadway design elements or access management technique on driver behavior.

Past safety studies (*Hallmark et al., 2015; Wu et al., 2015; Victor et al., 2014; Hallmark et al., 2013; Hutton et al., 2014; Lin et al., 2016; Camey et al., 2015; Dingus et al., 2016; Simmons et al., 2016; Harlan et al., 2016; and Kidd et al., 2015*) suggested the use of NDS data can quantify different factors' impacts on driver behavior and crash risk, compared with data from driving simulator, field experiments, or crash reports. These studies addressed road departure, offset left-turn lanes, driver inattention, rear-end crashes on congested freeways, etc.

NDS data may fill the gaps in crash data analysis with real-time continuous naturalistic driving data. In this section, the analysis of NDS data was utilized to investigate the effects of intersection balances at parclo interchange terminals on driver behavior that may result in less wrong-way entries at interchange terminals. Corresponding recommendations can be made to validate the current best practice on the intersection balance guidelines developed by WSDOT.

CHAPTER 3 IN-DEPTH INVESTIGATION OF WRONG-WAY CRASHES IN ALABAMA

3.1 Introduction

Wrong-way driving (WWD) crashes on divided highways is a critical threat to traffic safety as the speeds on these roads are so high that surviving this sort of crash is very unlikely. This type of crash occurs relatively infrequently, but the consequences are devastating. When comparing WWD fatal crashes with overall fatal crashes on divided highways, the number of WWD crashes appeared to remain fairly constant while the total number of fatal crashes within the same time period declined substantially. In Alabama (AL) from 2009 to 2013, WWD crashes on divided highways resulted in 16 more fatalities per 100 fatal crashes than non-WWD crashes. Approximately 5% of all fatal crashes were due to WWD even though WWD crashes accounted for 0.23% of all crashes on divided highways. Approximately 68.8% of the WWD vehicles became disabled after crashes, while this percentage for other divided-highway crashes was only 37%, resulting in more vehicles being towed after WWD crashes (73.2% of WWD vehicles vs. 57.5% of others). Also, 95% of WWD crashes involved two or more vehicles, while the average was 82% for all crashes on divided highways. However, few studies were done on WWD on divided highways.

Thus, an in-depth investigation of WWD crashes exclusively on divided highways was conducted. Extensive efforts on WWD crash data and field data collection were elaborated in Section 2. Contributing factors were identified by data-driven analyses in Section 3. Countermeasures were developed in Section 4. Section 5 provides conclusions and recommendations.

3.2. Data Collection

3.2.1 Crash Data

Five-year (2009 -2013) crash data were collected on divided highways with hard-copy crash reports and electronic data by the Alabama Department of Transportation's Office of Safety Operations and the Critical Analysis Reporting Environment (CARE). Altogether, 112 WWD crashes were collected on about 900 miles of divided highways in Alabama.

3.2.2 Wrong-Way Entry Points

Coordinates of the 112 WWD crash locations were extracted. Possible WWD entry points were estimated with Geographic Information System (GIS) capabilities of Google Earth and cross-checked with the crash reports. Special attentions were paid to crash narratives, collision diagrams, and crash circumstances. Hence, 112 WWD entry points on divided highways were identified (See Table 6). 5 recorded and 99 estimated wrong-way entries were at median-crossroad intersections. 5 made left turns at an access point with no median opening. 2 entered wrong ways by undeliberate/improper lane change. 1 was interchange related.

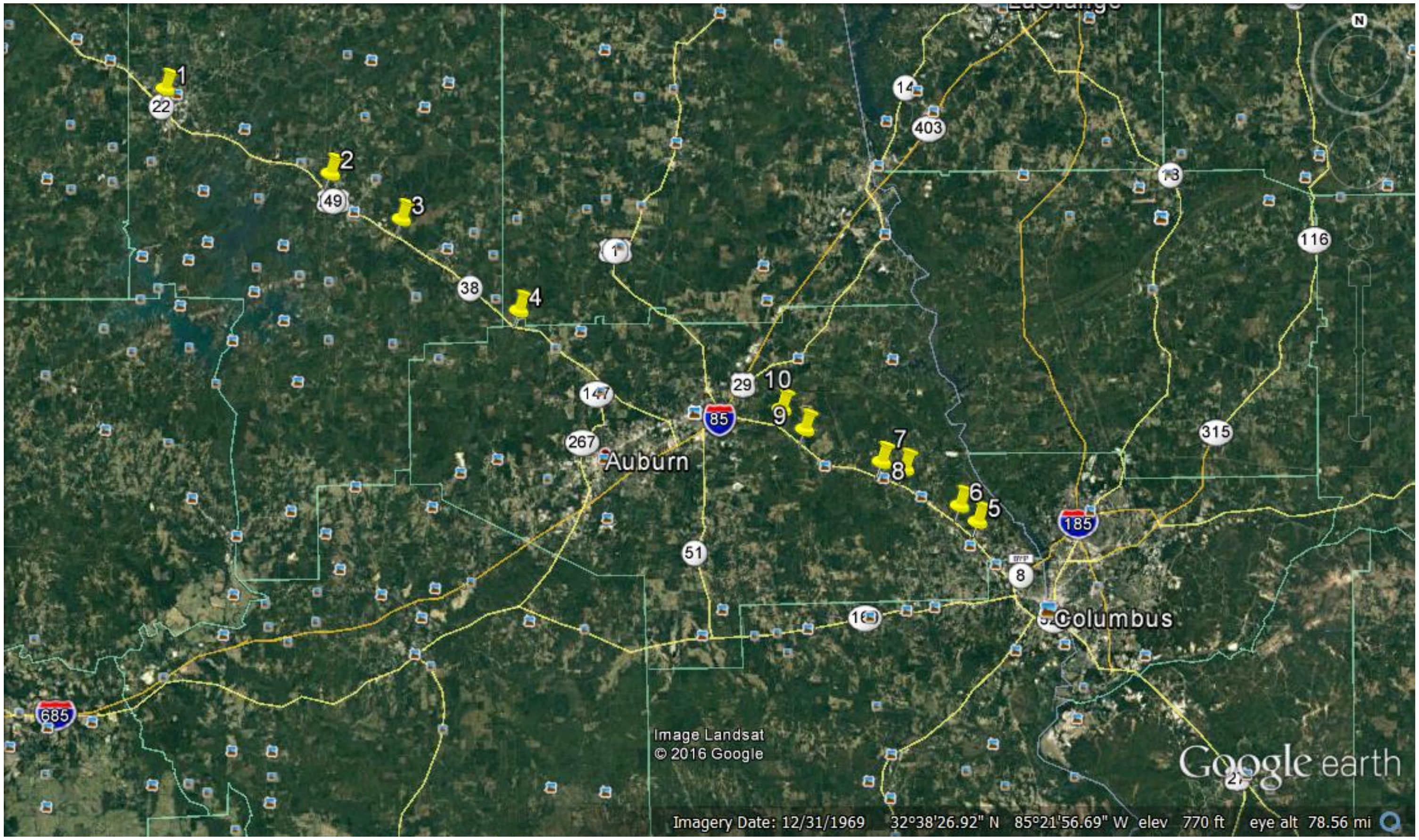
TABLE 6 Wrong-Way Entry Points

Category		Number
WWD Crashes with Known Entry Points	Recorded Entry at a Median Opening	5
	Entry from the Parking Lot of a Gas Station/Business Area (*Left-Turn at an Access Point with No Median Opening*)	5
	Entry by Undeliberate Lane Change *(Distracted and DUI)*	1
	Entry by Improper Lane Use *(Travelling/Speeding Through A Median Opening on the Two-Way Turn Lane)*	1
WWD Crashes with Unknown Entry Points	Interchange Related (*3 Interchanges but No Median Opening Nearby*)	1
	Estimated Entry at a Median Opening	99

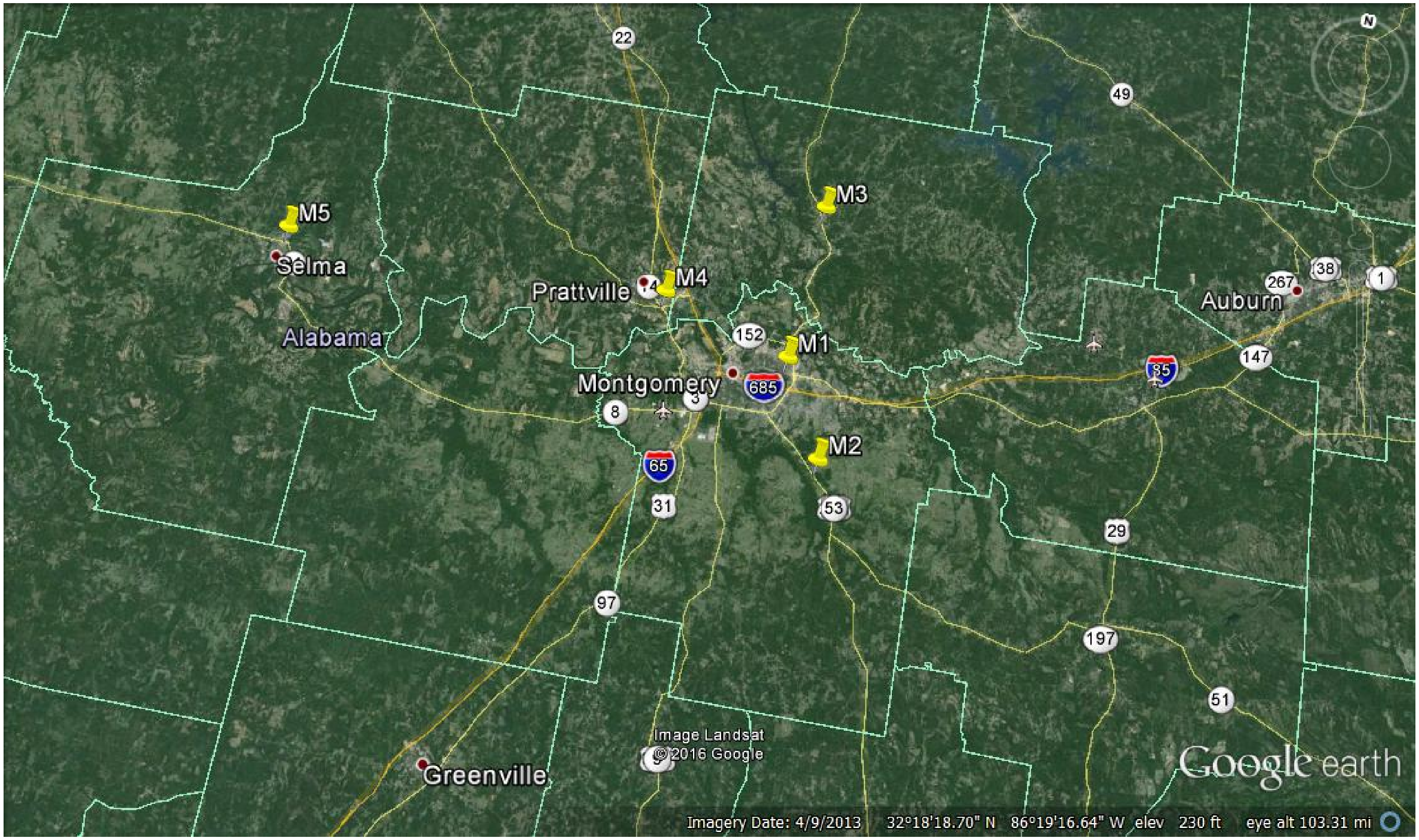
3.2.3 Field Observation and Inspection Data

Crash reports contain information about the overall crash scene, driver, and vehicle, but lack essential site-specific features of entry points that may either confuse drivers and lead to a WWD maneuver or help mitigate the issue, which may be of equal if not greater importance. Thus, field reviews were made to identify other relevant factors and inspect physical characteristics of wrong-way entry points to supplement crash data.

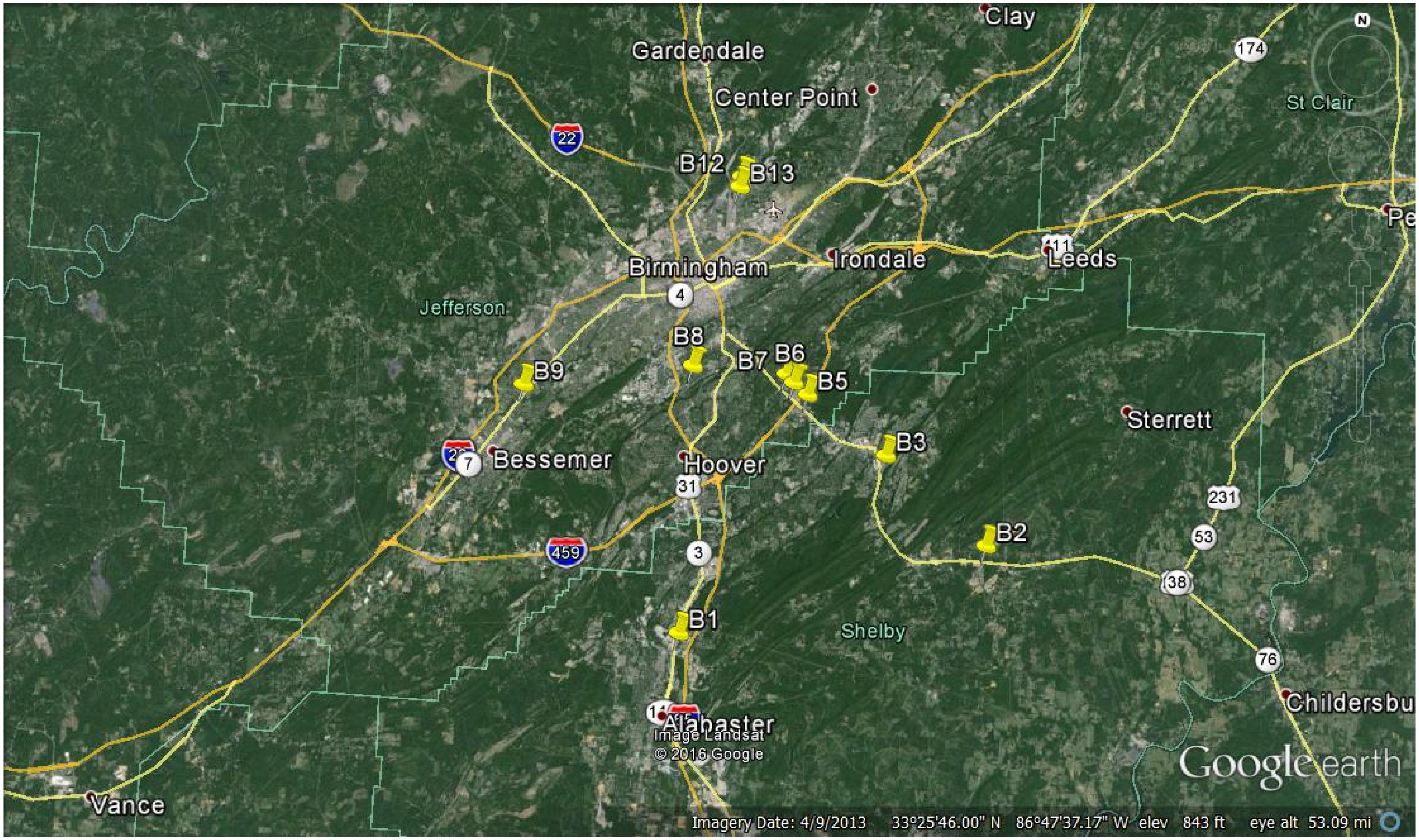
Thirty-four sites with median widths distributed among 0-120 feet were randomly selected from the identified WWD entry points on Alabama divided highways to conduct comprehensive field review. Field trips were made during nighttime between 8:00 P.M. and 4:00 A.M. on April 4, April 5, May 15, May 18, and May 19, 2015, including 10 locations near Auburn along a 63.9 mile segment from Milepost 120.249 to Milepost 135.605 on U.S. Highway 280, 5 locations in Montgomery, 13 in Birmingham, and 6 in Phenix City. The detailed map of these locations can be found in Figure 4 below.



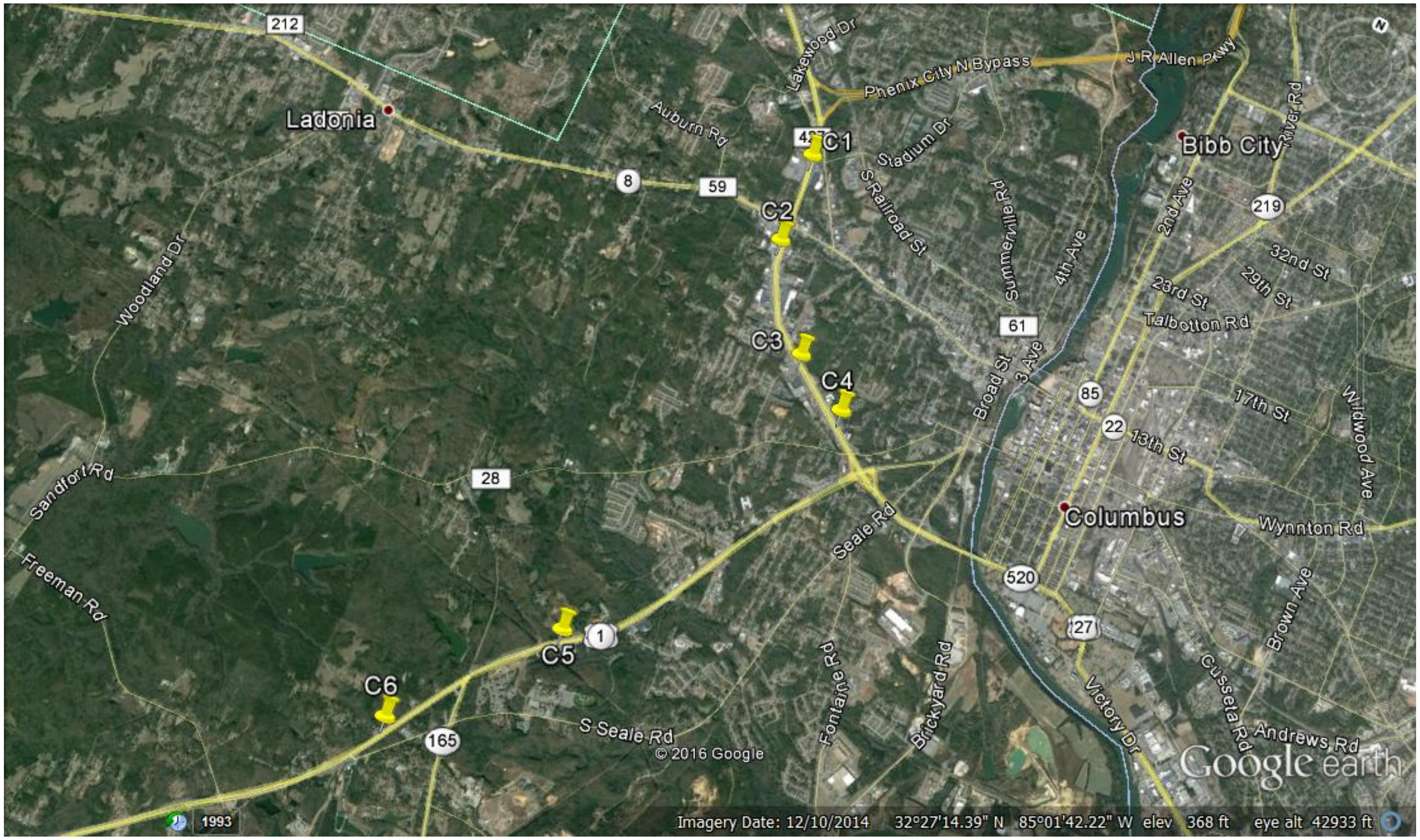
(a) Field Observation Locations near Auburn AL, along a 63.9 mile segment on U.S. Highway 280 (10 Locations)



(b) Field Observation Locations in Montgomery, AL (5 Locations)



(c) Field Observation Locations in Birmingham, AL (13 Locations)



(d) Field Observation Locations in Phenix City, AL (6 Locations)

FIGURE 4 Field review locations

3.2.4 Geometric Data

Detailed geometric data (Figure 5) of study locations were collected on fields and cross checked using GIS capabilities of Google Earth, including S: Available sight distance (feet), M: Median width measured per the MUTCD definition (feet), W: Width of the lanes in one direction on the adjacent side of the stop bar (feet), D: Distance from stop bar to the adjacent lane (feet), O: Width of the opposite side roadways (feet), L: Width of the entire cross-section of the median opening (feet), Ø: Upswept angle of headlight beam from horizontal (typically 1 degree), E_2: elevation of the ending point of the roadway segment, $E_{1:}$ elevation of the starting point of the roadway segment, G/G1/G2: Algebraic Grade Change in percent (%). Grades were calculated by equation (1):

$$G\ (\%) = \frac{E_2 - E_1}{L} * 100 \qquad (1)$$

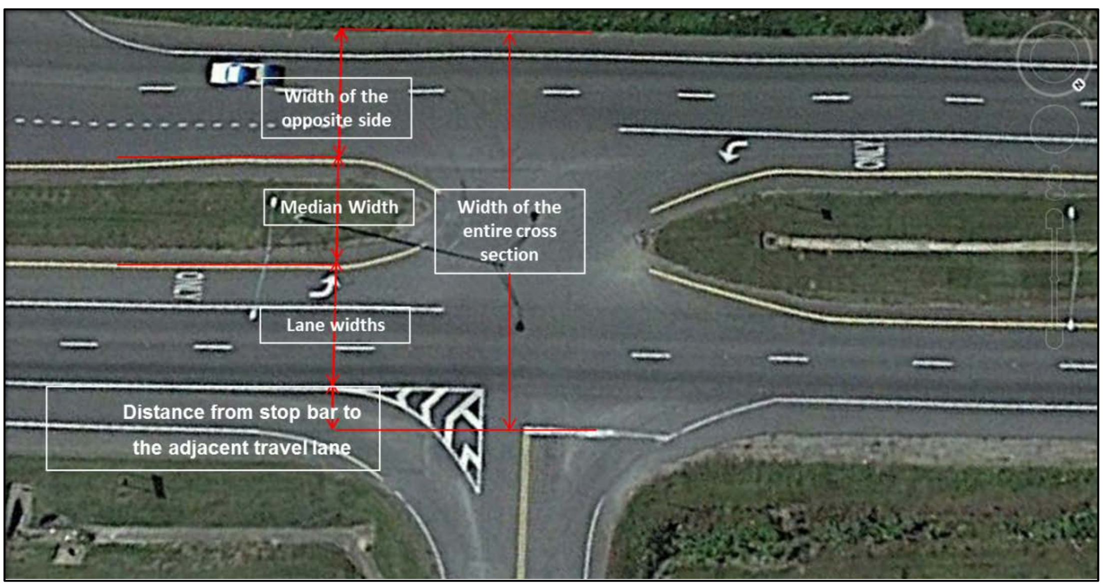


FIGURE 5 Diagram of the measured parameters

Furthermore, the actual sight distance at driver's eye height in a passenger car was checked in the field. Relevant data such as intersection controls, advanced signage, vertical curvature, and intersection conspicuity, which could not be clearly identified from online maps, were collected during field reviews. For each intersection approach, several more attributes were collected: 1) The presence, type, and amount of traffic control devices (TCDs), which were grouped into five types (conventional guide signage, freeway-style [larger] guide signage, overhead signage, route number signage, and warning signage); 2) The distance for intersection recognizability: the distance to the intersection at which there are obvious indications of the intersection presence, and if it is shorter than drivers' sight distance; 3) Median type and width; and 4) Grades and vertical curvature (e.g., upgrade or downgrade, crest or sag vertical curve, etc.).

These data were documented by photos taken in the field. Figure 6 gives examples of the driver front view, left side view, and right side view with vehicle headlight illumination only at one of the study sites. From the front view in Figure 6 (a), observed TCDs in the roadway environment consisted of a STOP sign, One-Way signs mounted on STOP sign and on the opposite side of the cross section, and different pavement markings. From the side views in Figures 6 (b) and (c), other types of TCDs can be seen, such as the Do Not Enter (DNE) sign, WW sign, Yield sign, etc.

(a)

(b) (c)

FIGURE 6 Examples of the Driver Front View (a), Left Side View (b), and Right Side View (c) on the Divided Highway

Recommended by the AASHTO Green Book (*2011*), vehicles' average headlight span is 160~180 feet and drivers' eye height in a passenger's car is 3.5 feet above the roadway. Field experiments were also conducted to verify the two specifications of the vehicle used in the experiment. The experimental vehicle was a 2010 Chrysler Sebring passenger car. The vehicle manual indicated that the headlights can reach approximately 160 feet during nighttime. Measurements were performed multiple times with a measurement wheel. The measured headlight span and driver's eye height were approximately 164 feet and 3.5 feet on average.

3.2.5 Sight Distance Determination

Field experiments were conducted to determine drivers' sight distances from crossroads at nighttime. When a driver is completing a crossing maneuver, there must be sufficient sight distance in both directions to cross the intersecting roadway and avoid approaching traffic. If the sight distance is shorter than the entire cross-section width, the segment should be treated as two separate intersections.

As current sight distance calculation methods in the AASHTO Green Book (*2011*) are not applicable to this problem, profiles of the entire cross section with drivers' sight in a passenger car were drawn for the 110 possible WWD entry median openings. Sight distance at the median opening was determined based on the projection of drivers' sight on the profile of the entire cross section with worst-case scenarios, i.e. sight distance with only headlight illumination at nighttime. The basic rationale behind this method is that if drivers making left turns or going through from side streets cannot see the far side main roadways, then it is necessary to treat the median opening as two intersections for WW-related TCD requirements. Given that most WWD crashes occurred at nighttime, the sight distance was only checked for nighttime conditions. Figure 7 illustrates an example. The results were compared with entire cross-section widths to check adequacy. Adequacies were also cross-checked in the fields.

Furthermore, for each intersection approach, attributes collected include: (a) the presence, type, and amount of TCDs; (b) distance of intersection recognizability: the distance to the intersection at which there are obvious indications of the intersection presence; (c) Median type and width; and (d) grades and vertical curvature.

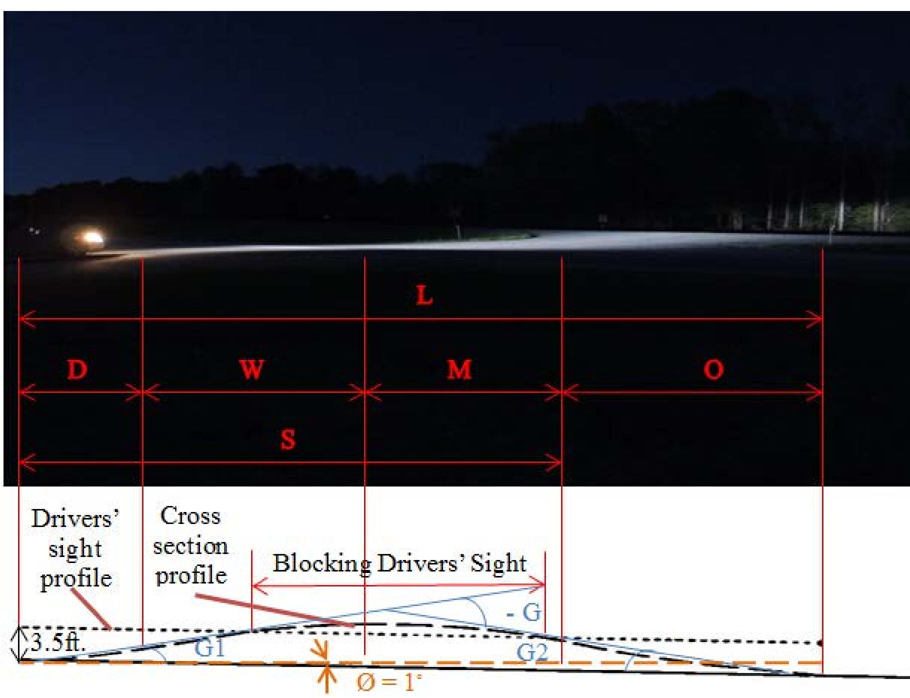


FIGURE 7 Example of profile drawing for median-crossroad intersections

According to literature review, besides geometric features, TCD conditions of WWD entry points and crash locations are also significant factors. A checklist (Figure 8) specifically for divided highways was, therefore, designed for field inspection. It was designed to examine that the existing TCDs were adequate to deter WWD at each study location, which include the visibility, number, and condition for all possible types of TCDs. This checklist serves as a tool to assess possible problematic locations identified for WWD prevention based on general issues in terms of sight distance, signage, pavement marking, and geometric design on multilane divided highways.

Divided Highway Wrong-Way Entry Nighttime Field Inspection Checklist

Inspector:				
Route Information:			Date	
Median Width:			Time	

SIGN	CHECK IF	YES	Number	CONDITION & COMMENTS
	Present			
	Can be seen by driver			
	Present			
	Can be seen by driver			
	Present at adjacent side			
	Present at opposed side/in median opening			
	Present at side road			
	Present at opposed side/in median opening			

PAVEMENT MARKNG	CHECK IF	YES	Number	CONDITION & COMMENTS
Stopping bars at side road	Present			
Stopping bars in median opening	Present			
Yellow lines in median opening	Present			

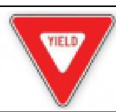
GEOMETRC DESIGN FEATURES	CHECK IF	YES	Number	CONDITION & COMMENTS
Raised curb/other special median	Present			
	Present			
	Present			
Farthest pavement marking seen by driver	Median edge line Pavement edge line Centerline Farther pavement edge line		N/A	

FIGURE 8 Field inspection checklist for divided highways

3.3 Contributing Factor Identification

3.3.1 Haddon Matrix Data Analysis

Haddon matrix, developed by William Haddon Jr., is a two-dimensional model commonly used to approach safety analysis at sites in a systematic fashion. One dimension is crash sequence, which is divided into pre-crash, crash, and post-crash. The other is crash factors categorized into human, vehicle/equipment, and environment. Each cell of the matrix illustrates risk or protective factors involved in the phase, where contributing factors, circumstances, and countermeasures can be identified. When completed, it provides insight into the range of possible safety issues and concerns due to human, vehicle, and roadway, as well as when to implement countermeasures.

In this study, Haddon matrices were developed for severe WWD crashes, i.e. fatal, A-injury (incapacitating), and B-injury (non-incapacitating) crashes. Accordingly, cumulative frequencies were calculated for each cell: pre-crash human, pre-crash vehicle, pre-crash environment, during-crash human, during-crash vehicle, during-crash environment, post-crash human, post-crash vehicle, and post-crash environment. Results were summarized for five groups (Fatal, A-injury, B-injury, Fatal and A-injury, and Fatal, A-injury, and B-Injury).

Regarding human factors, drivers aged above 65 and 35-44 have the largest frequencies, 38% and 24%, respectively, for fatal crashes and B-injury. The prevalent age groups for A-injury were 25-34 and 35-44, both 23%. Male drivers were over-represented for all crashes, 69% of fatal, and 59% of both A-injury and B-injury. In all severe crashes, DUI was the most common known driver condition. Traveling wrong way/wrong side was the most ubiquitous driver contributing circumstance. This could explain why driver sight distance was restricted.

Concerning vehicle factors, most vehicles involved were passenger cars, 62% for fatal, 50% for A-injury, and 71% for B-injury. The prevalent maneuver for both the at-fault vehicle and other vehicles was essentially straight, which lead to the fact that the largest amounts of crashes are head-on, 93%, 72%, and 55% of fatal crashes, A-injury crashes, and B-injury, respectively. Air bags had been deployed in most WWD crashes, another indicator of the crash severity. Seatbelt Usage shows a decreasing trend with increasing of crash severity, 62% for fatal, 64% for A-injury, and 82% for B-injury, which shows the active role of seatbelt use in reducing injury for vehicle occupants during crash.

For environmental factors, most crashes of all severities happened on dry pavement surfaces, 92%, 86%, and 71% for fatal, A-injury, and B-injury crashes. Darkness/Road Not Lit was the common condition for 69%, 55%, and 71% of fatal, A-injury, and B-injury crashes. Additionally, most WWD crashes were unrelated with Work Zones. The weather condition was clear. Combined with human factors, the finding indicates DUI driving is more likely to occur during nighttime when the visibility of roadways and TCDs is compromised. The more detailed frequencies can be found in Appendix A: Contributing Factor Frequency

3.3.2 Correlation Analysis

Furthermore, a correlation analysis was conducted based on the time and severity of each crash and the age and physical condition of each driver. The factors of interest are Young Drivers, Older Drivers, DUI Drivers, Night Crashes, Male Drivers, Weekend Crashes, and Fatal/Injury Crashes. Each factor was denoted as a binary variable for all 112 crashes. Thus, the array of each factor contains 112 values. The analysis assists investigators in connecting contributing factors to the time, the driver age, and the physical condition of the crashes. The closer the results are to a value of one, the more relation the two factors share and the stronger the correlation. If the results indicate a negative relation, one factor increases as the other decreases. The correlation analyses results are presented in Table 7. For example, the correlation between "DUI Drivers" and "Night Crashes" is a significant positive number, which means that, once a WWD crash happened during nighttime, it is very likely to involve DUI drivers. There is a negative correlation between "Older Drivers" and "Night Crashes," and it indicates that older drivers are relatively unlikely to lead to a WWD crash at night. Significant correlations have been found between "Young Drivers" and "Weekend Crashes," "Older Drivers" and "Night Crashes," "Older Drivers" and "Male Drivers," "Older Drivers" and "Fatal/Injury Crashes," "DUI Drivers" and "Night Crashes," "DUI Drivers" and "Male Drivers," "Night Crashes" and "Male Drivers," and "Weekend Crashes" and "Fatal/Injury Crashes."

TABLE 7 Correlation Analysis between Contributing Factors

	Young Drivers	Older Drivers	DUI Drivers	Night Crashes	Male Drivers	Weekend Crashes	Fatal/Injury Crashes
Young Drivers	1						
Older Drivers	-0.277*	1.00					
DUI Drivers	-0.042	**-0.32**	1				
Night Crashes	0.072	**-0.272**	**0.240**	1.00			
Male Drivers	0.051	**-0.307**	**0.234**	**0.119**	1.00		
Weekend Crashes	**0.130**	-0.059	0.082	-0.028	0.032	1.00	
Fatal/Injury Crashes	-0.023	0.215	**-0.143**	0.028	-0.064	**-0.147**	1.00

Note: *there is no practical meaning between older drivers and young drivers.

3.4 Field Inspections and Countermeasures

3.4.1 General Issues

Of the 34 study locations, with median widths that ranged from 7.6 to 118.8 ft., six were treated as two separate intersections for TCD installation, while twenty (including three that were treated as two intersections) were subject to the current MUTCD criteria (*2009*), i.e., when median width ≥ 30 ft. Six (including one treated as two intersections) used the method developed in the present study.

Moreover, five (two narrow medians and three wide medians) have partial MUTCD required Keep Right, ONE WAY, and pavement marking; six have MUTCD required TCDs; nineteen have no DNE and WW signs; nine wide medians have partial required DNE, WW, and marking; fourteen (eight narrow medians and six wide medians) have no any TCD. Since the MUTCD provides the minimum standards for TCDs, 53% ((6+3+9)/34) of the study locations lack necessary wrong-way signing; 30% ((2+8)/34) lack necessary one-way signing.

General issues regarding signing (including DNE, WW, ONE WAY, No Left/Right Turn, and Keep Right), pavement marking (including double yellow lines, white edge line, stop line on crossroads, stop line on median roads, Left/Right Turn Only Arrow, and Lane-line Extensions), and geometric designs are detailed in Table 8. Figure 10 - 14 shows selected examples of issues in different categories.

TABLE 8 Summary of General Issues

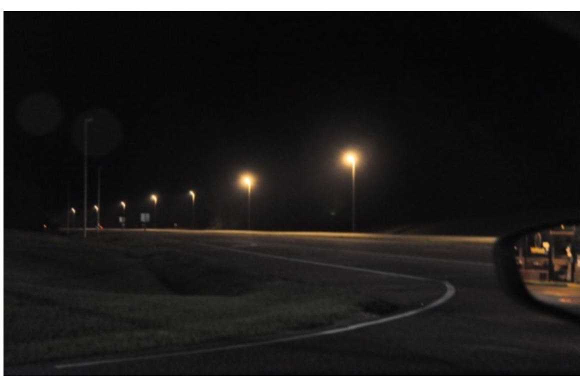
Category	Issues
Signing	Visibility can be improved for nighttime and low visibility conditions
	Angle of signs needs to be adjusted: some DNE signs do not face the potential WW drivers
	Absence of required and optional signs at some locations
	Some DNE signs are placed so far from the nose of the median that they can be observed only after drivers enter the wrong way
	The heights of some DNE and WW signs are above the reach of crossroad drivers' headlights illumination
Pavement Marking	Visibility can be improved
	Absence of required and optional pavement markings such as double yellow line, white edge line, and stop line on crossroads and median roads
	Faded pavement markings
	Confusing pavement markings
	Markings of adjacent crossroads were extended too far
	Lack of directional pavement markings to guide the large-turning radii drivers at wide median opening locations
Geometric Design	The elevation of crossroads should be equal to or great than that of the divided highway
	Access control at gas station or service areas, e.g. channelized driveway, roadway information signage, closing the driveway, etc.

	Sight obstructions at median openings
	Unconventional intersection layouts

3.4.1.1 Traffic Sign

Figure 9a gives an example of absence of required and optional signs at a median opening. No signage is present for both directions of the divided highway. Figure 9b shows an example of an improperly oriented sign. The DNE sign is parallel to the divided highways, not facing the potential right-turn WW drivers. Figure 9c shows an example of DNE signs placed so far away and can be observed only after drivers enter the wrong way.

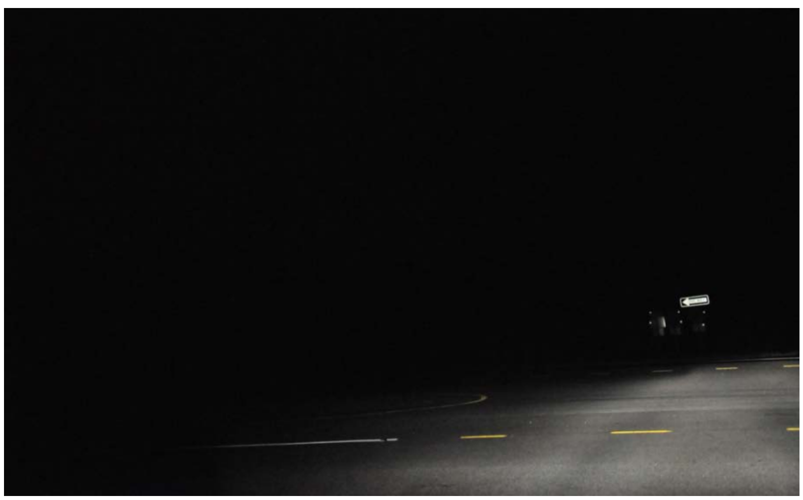
(a-1) Front View



(a-2) Left-Side View **(a-3) Right-Side View**

(a) Examples of sign absence at a median opening in AL

(b) Example of an improperly oriented sign in AL



(c) Example of DO NOT ENTER signs placed too far from the nose of the median in AL

FIGURE 9 Examples of traffic signs from field review

3.4.1.2 Pavement Marking

Figure 10 shows an example that pavement marking of the lanes and median edges are faded at a median opening, which makes the roadway layout less recognizable for the drivers on crossroads.

(a) Left-Side View **(b) Right-Side View**

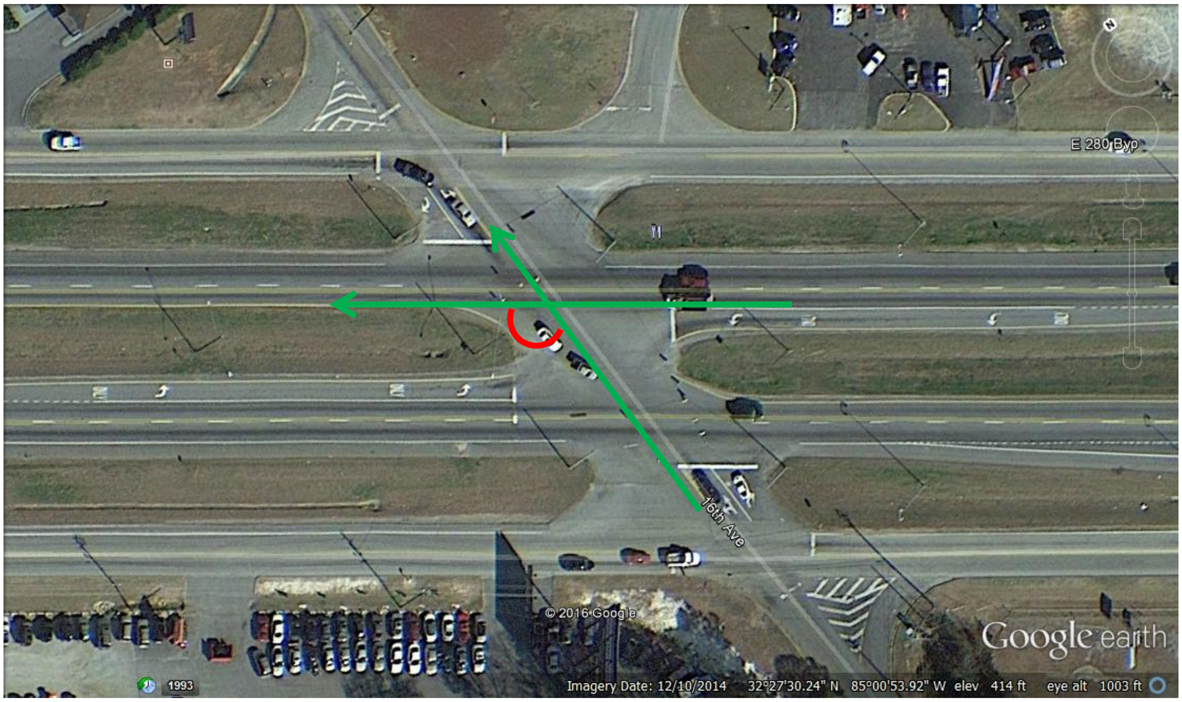
FIGURE 10 Example of faded pavement marking at a median opening in AL

3.4.1.3 Geometric Design

Figure 11a shows a raised island presence at the median opening, which could block driver sight of the far side of the divided highway at nighttime. Figure 11b shows an example of big elevation changes at a median opening. The elevation of the divided highway is higher than that of the crossroad, which makes it hard for drivers to perceive the presence of the far side divided highways, especially at nighttime. Figure 11c shows an example of skewed intersection at a divided highway-crossroad intersection. Such layouts are frequently confusing and may encourage WWD based on the past studies.

(a) Example of a raised island at a median opening in AL

(b) Example of big elevation changes at a median opening in AL



(c) Example of non-90 degree intersection angles in AL

FIGURE 11 Examples of geometric design from field review

3.4.2 General Countermeasures

Measures that improve the driver's visibility and perception of access points onto divided highways would potentially decrease wrong-way movements. Improvements of traffic signs and pavement markings are cheaper and easier to implement and are recommended for all. Optional TCDs are also recommended at WWD vulnerable locations. Geometric features and ITS applications require tremendous costs for installation and operation/maintenance, recommended only for locations with wrong-way crash/incident history. The effectiveness of WWD countermeasures was considered by whether the countermeasures could deter wrong-way movements effectively and impact the right-way motorists (*Zhou et al., 2012*). Table 9 summarizes detailed countermeasures, which are either standards by the MUTCD or AASHTO Green Book or proven effective based on literature review and this study.

TABLE 9 Summary of General Countermeasures

Category	Issues
Signing	Supplement the MUTCD guideline on TCDs at median openings with the new method developed in this study. If drivers' sight distance is not sufficient, WW-related TCDs should follow the guidelines for two separate intersections
	Increasing the visibility of signs by using following techniques: o Internally illuminated signposts using flashing, internally illuminated signs. o Add small LED units along the sign's borders to attract drivers' attention. o Affix red retro-reflective tape to the signposts to enhance nighttime visibility, particularly for those who are impaired, disoriented, or confused. o Orient the existing signs towards potential WWD drivers o Lower the heights of DNE and WW signs o Place at least one pair of DNE and WW sign to fall within the area covered by vehicles' headlights
Supplemental/Optional Sign	Provide signage to help drivers on the crossroad approaches recognize that left turns into the near roadway of the divided highway are not permitted (See Figure 13 for an example). This may include One-Way signs, No Left Turn sign, WW signs, and Divided Highway plaques
	Supplemental items such as placards, flashing beacons, or flags can be added to DNE and/or WW signs as an enhancement to the traditional approach
	Use the optional Keep Right sign and/or ONE WAY sign at median openings
	Use No Right Turn sign at median openings
	Use Divided Highway signs (R6-3 or R6-3a in the MUTCD) to provide intersection geometry information to the drivers entering a multilane divided highway
	Install another set of "WW, DNE" signs farther down the wrong way at problem median-crossroad intersections to give drivers a second chance to realize their mistake
Pavement Marking	Install markings such as yellow line, white edge line, and stop line on crossroads and median openings (See Figure 13 for an example), based on the MUTCD for intersection treatments
	Increase the visibility of pavement markings by adding retroreflective raised pavement markers
	Avoid confusing pavement markings such as too-far extended two-way turn lanes, painted median between lanes in the same direction
	Use lane-use arrow on each lane of the divided highway upstream of the crossroad intersection
Supplemental/Optional Pavement Marking	Add optional WW arrows and Stop Line equipped with Raised Pavement Markers
	Delineate of the nose of the median by curb and paint (Yellow). The median should be distinct to aid the driver in understanding the intersection layout and function. Distinctiveness can be achieved by raising and coloring the median. Medians do not have curbs or pavement edge markings around the nose of the median could be very inconspicuous at nighttime for old drivers and slightly impaired drivers
Geometric Design	Clear sight obstructions at the median opening that limit the view of the far-side roadway
	Design the elevation of crossroads equal to or great than that of the divided highway to give drivers on crossroads a clear view of both directions of highways

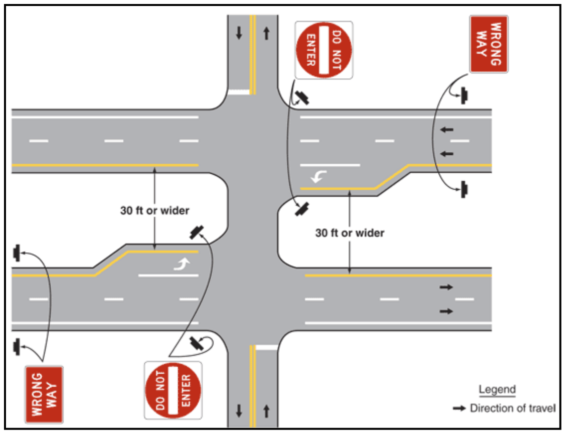
	Avoid skew angles of intersection as well as unconventional intersection layouts
	Use channelization and barrier curbs on crossroads/business area driveways to direct traffic in the right direction and block WW movements
Median Type, Design, and Modification	Replacing painted median with raised median to better direct traffic movements
Access Management	Use access management techniques such as channelized islands, and provide exclusive U-turn bays for in direct left turns from driveways at service areas
Others	Provide spotlighting to make roadways conspicuous to assist old or impaired drivers
	Whenever possible, divided highway intersections with wider medians and/or big grade changes should be lighted to assist drivers in seeing both sides of the major road and recognizing it as a divided highway
	Adding data-driven DUI checkpoints
	Use Radio, Autonomous Vehicle, Connected Vehicle, Vehicle to Vehicle (V2V), and Vehicle to Infrastructure (V2I) to warn right-way drivers of oncoming WW drivers
	Coordinate with the primary 911 public safety answering points to share information on reports of WW movements on highway facilities
	Use Prototype WWD Detection System/Video Surveillance and Detection System.

The definition of an intersection in Section 1A.13 of the 2009 MUTCD indicates that crossings of two roadways 30 feet or more apart shall be considered two separate intersections. Figure 13 (a) shows the recommended WW signing for divided highways with median widths of 30 feet or wider. When median width is 30 feet or greater, the DNE sign and WW sign should be placed directly in view of a road user at the point where a road user could wrongly enter a divided highway. The WW sign is used as a supplement to physically discourage or prevent WW entry at intersections of divided highways. If used, it should be placed farther downstream than the DNE sign on the divided highway. In Figure 12 (a), the DNE signs are placed at a 45-degree angle facing the potential WW drivers (*FHWA, 2009*).

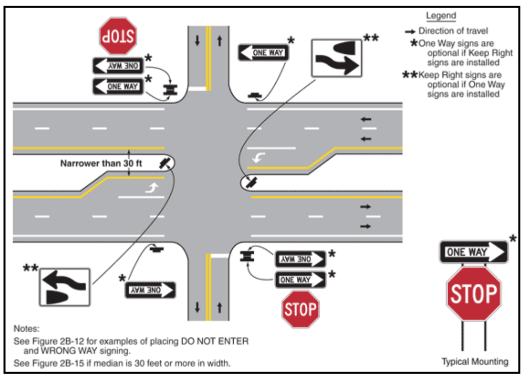
Figure 12 (b) and 12 (c) replicate Figures 2B-16 and 2B-15 in the MUTCD illustrating the recommended One Way signing at divided highway intersections with narrow and wide (30 feet or wider) medians, respectively (*FHWA, 2009*).

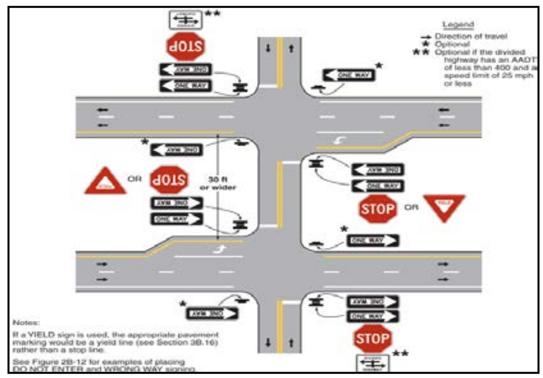
The MUTCD states "*At an intersection with a divided highway that has a median width at the intersection itself of less than 30 feet, Keep Right signs and/or ONE WAY signs shall be* *installed. If Keep Right signs are installed, they shall be placed as close as practical to the approach ends of the medians and shall be visible to traffic on the divided highway and each crossroad approach. If ONE WAY signs are installed, they shall be placed on the near right and far left corners of the intersection and shall be visible to each crossroad approach*" (*FHWA, 2009*). As shown in Figure 13, no control (yield, stop, or signal) is provided for the interior approaches when the junction functions as a single intersection when the median is narrower than 30 feet. Similar to intersections of undivided highways, the basic assumption is left-turning vehicles from opposing directions do not cross paths (i.e., turn in front of each other).

Meanwhile, "*At an intersection with a divided highway that has a median width at the intersection itself of 30 feet or more, ONE WAY signs shall be placed, visible to each crossroad approach, on the near right and far left corners of each intersection with the directional roadways*" (*FHWA, 2009*). As shown in Figure 12 (c), interior control in the median is provided when the median is 30 feet or wider and the location functions as two separate intersections. In this case, opposing left turns from the divided highway cross paths (i.e., turn behind one another) and the interior of the divided highway is treated as a roadway rather than as part of the intersection. This leads to the need for additional signing at each separate intersection.



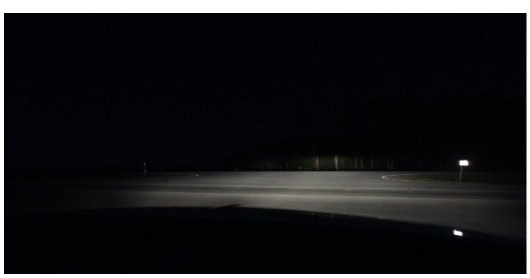
(a)





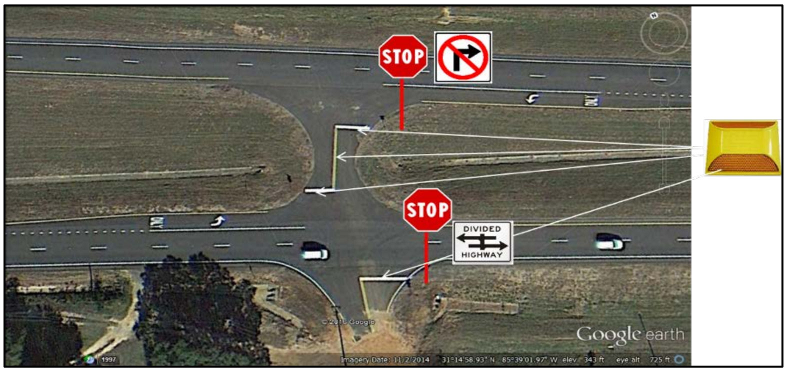
(b) **(c)**

FIGURE 12 WW Signing for Divided Highways with Median Widths of 30 Feet or Wider (a) and One Way Signing at Divided Highway Intersections with Narrow (b) and Wide Median (c) (*FHWA 2009*)



(a) Current Condition

(b) Suggested Treatments



(c) Suggested Countermeasures

FIGURE 13 Example of suggested TCDs to be Installed at a median opening in AL

3.4.3 Case Study

Of the sites inspected in this study, the following six divided highway-crossroad intersections, recorded as known entry points of WWD crashes in the study period, were used for case study. The first five were wrong-way entries from crossroads at nighttime. The sixth WWD entry resulted from undeliberate lane change at daytime. Three occurred on weekends. Four were in rural areas. Four WWD entries were made by unimpaired drivers and 2 by drunk drivers. All drivers are male, including five aged over 65 and one aged 25-34.

3.4.3.1Case 1

One wrong-way entry by an unimpaired driver was recorded at this signalized intersection (Figure 14) with median width of 34 feet. Deficiencies identified are:

a. Large turning radius for left turning vehicles,

b. Lack of pavement marking to guide large left turning movements,

c. Painted median was used to separate through and left turn lanes.

Recommended countermeasures are to:

a. Add the elephant pavement marking to guide left turns,

b. Reduce the width to median nose to increase the width of receiving lanes.

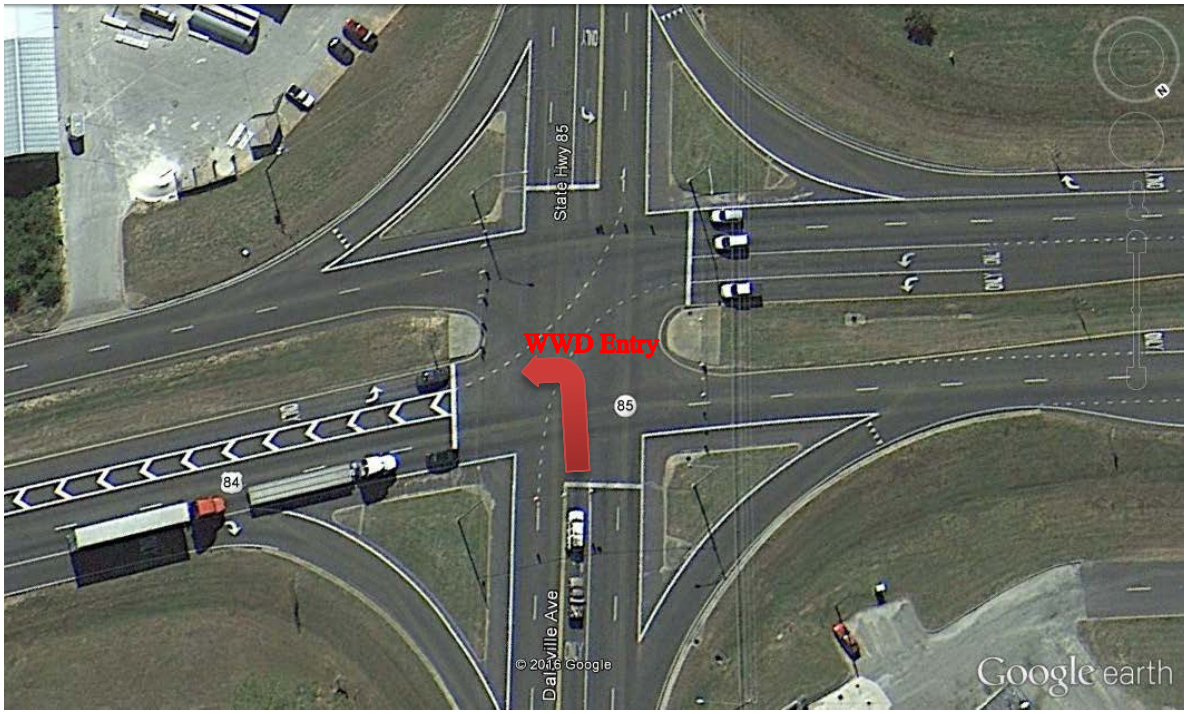


FIGURE 14 Signalized Intersection of US Highway 84 E (Divided Highway) and AL Highway 85 (Crossroad) in AL

3.4.3.2 Case 2

One wrong-way entry by an unimpaired driver was recorded at this unsignalized intersection (Figure 15) with median width of 20 feet. Deficiencies identified are:

a. Both sides of the divided highways have no DNE or WW signs;

b. No supplemental signs, i.e. Keep Right, ONE WAY, or Divided Highway signs in the median or at crossroad stop line;

c. No STOP sign at the crossroad stop bar.

Recommended countermeasures are:

a. Adding lane-use arrows on both directions of major highways,

b. Adding STOP sign at the driveway,

c. Adding KEPT RIGHT sign at the median.

FIGURE 15 Unsignalized Intersection of US Highway 31 (Divided Highway) and IGA store parking lot driveway (Crossroad) in AL

3.4.3.3 Case 3

One wrong-way entry by an unimpaired driver was recorded at this unsignalized intersection (Figure 16) with median width of 77 feet. Deficiencies identified are:

a. Both sides of the divided highways have no WW signs,

b. DNE signs are not visible, too far away from the intersection,

c. No supplemental signs, i.e. Keep Right, ONE WAY, or Divided Highway signs at the median opening,

d. Faded pavement markings,

e. Darkness/no lighting exists.

Recommended countermeasures are:

a. Treating the median opening as two separate intersections,

b. Adding lane use arrow on both sides of major roads.

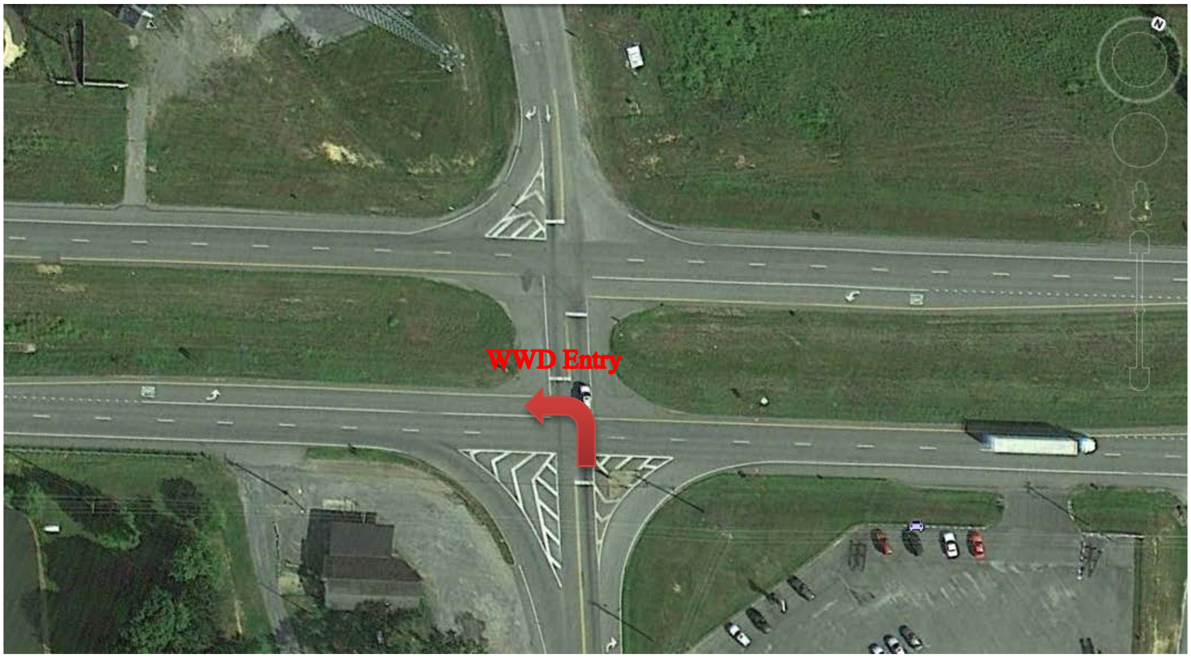


FIGURE 16 Unsignalized Intersection of AL Highway 75 (Divided Highway) and AL Highway 68 (Crossroad) in AL

3.4.3.4 Case 4

One wrong-way entry by a drunk driver was recorded at this unsignalized intersection (Figure 17) with median width of 30 feet. Deficiencies identified are:

a. No DNE or WW signs,

b. No supplemental signs, i.e. Keep Right, ONE WAY, or Divided Highway signs at the median opening,

c. No STOP sign at the crossroad stop bar,

d. Darkness/no lighting.

Recommended countermeasures are:

a. Adding STOP sign at the crossroad,

c. Improving WW-related signs,

d. Adding lane-use arrows on both directions of major highways.

FIGURE 17 Unsignalized Intersection of US Highway 231 (Divided Highway) and Private Property Driveway (Crossroad) in AL

3.4.3.5 Case 5

One wrong-way entry by an unimpaired driver was recorded at this unsignalized intersection (Figure 18) with median width of 18 feet. Deficiencies identified are:

a. No DNE or WW signs,

b. No roadway information signs, i.e. Keep Right, ONE WAY, or Divided Highway signs in the median or at the crossroad stop line,

c. No STOP sign at the crossroad stop bar,

d. Darkness/no lighting exists at nighttime.

Recommended countermeasures are:

a. Adding STOP sign at the crossroad,

b. Improving WW-related signs,

c. Adding lane use through arrow on both directions of major highways.

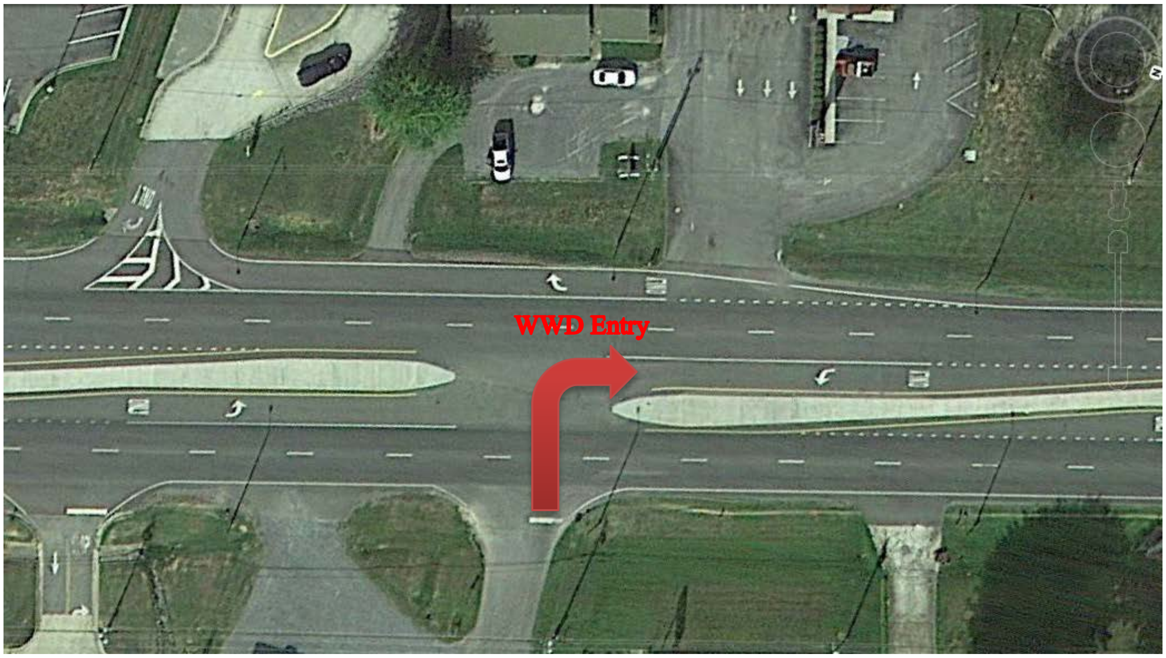


FIGURE 18 Unsignalized Intersection of US Highway 431 S (Divided Highway) and Byron Ave. (Crossroad) in AL

3.4.3.6 Case 6

One wrong-way entry was made by a drunk driver at the end of a two-way left turn lane at the intersection (Figure 19) with median width of 12 feet. Deficiencies identified are:

a. The two-way left-turn lane extended too far into the median road,

b. Painted median is not effective enough to stop WWD,

c. No signs to discourage WWD.

Recommended countermeasures are:

a. Shortening the two-way left-turn lane,

b. Installing raised curb median instead of painted median.

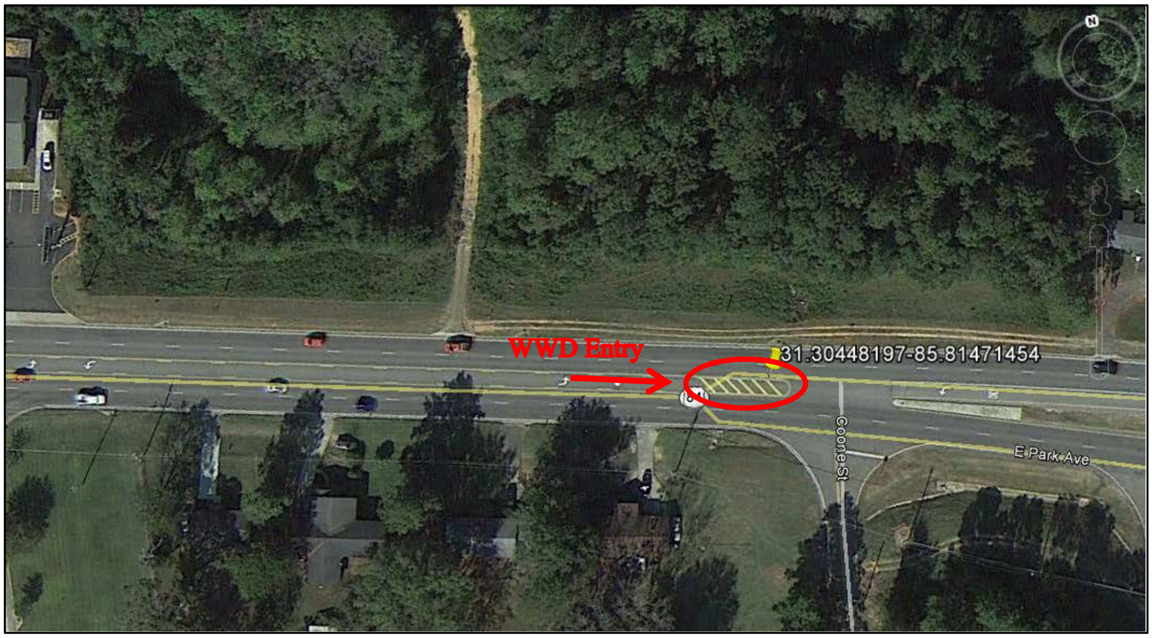


FIGURE 19 Unsignalized Intersection of East Park Avenue (Divided Highway) and Coone Street (Crossroad) in AL

3.5 Conclusions and Recommendations

3.5.1 Conclusions

This study developed and refined knowledge of the contributing factors and filled the gap of identifying countermeasures for WWD on divided highways, especially for Alabama. It was completed through an in-depth investigation of WWD crashes exclusively on divided highways. Extensive efforts were made on WWD crash analyses using the Haddon Matrix and field observations. General issues, contributing factors, and countermeasures were developed accordingly.

Haddon matrix analyses were conducted for individual crashes to identify the contributing factors regarding human, vehicle, and environmental factors in crash casual sequences. Cumulative results were summarized and concluded older drivers, male drivers, being under the influence of alcohol/drugs, driving passenger cars, and darkness were found as dominated contributing factors with highest frequency in severe (fatal, A-injury, and B-injury) crashes. The active roles of seatbelt and airbag use are shown in providing safety for the vehicle occupants during crash. Furthermore, correlation analysis indicated significant correlations between “Young Drivers” and “Weekend Crashes”, “Older Drivers” and “DUI Drivers”, “Older Drivers” and “Night Crashes”, “Older Drivers” and “Male Drivers”, “Older Drivers” and “Fatal/Injury Crashes”, “DUI Drivers” and “Night Crashes”, “DUI Drivers” and “Male Drivers”, “Night Crashes” and “Male Drivers”, and “Weekend Crashes” and “Fatal/Injury Crashes”.

WWD entry points were estimated with GIS capabilities of Google Earth and cross-checked with the crash reports for all 112 WWD crashes on divided highways in a five-year period in Alabama. Median-crossroad profiles were developed with geometric data collected for all 110 identified WWD entry points to determine sight distance of drivers on crossroads. The rationale behind this method is that if left-turn or through drivers from crossroads cannot see the far-side roadways at in dark conditions, it is necessary to treat the median opening as two intersections to prevent WWD.

Field inspections were conducted at 34 selected WWD crash locations and entry points with median widths ranging from 7.55 to 118.80 feet. A new inspection checklist was designed exclusively for divided highways to evaluate if current TCDs are sufficient to deter wrong-way movements regarding types, amounts, and conditions of traffic signing, pavement marking, and geometric features. Issues were identified. Corresponding countermeasures were developed. Both were summarized in tables for the ease of implementation to traffic agencies. Additionally, six divided highway-crossroad intersections, where WWD entries were recorded, were used for case studies to identify site-specific deficiencies and treatments.

3.5.2 Recommendations

The study developed various countermeasures of WWD crashes on divided highways. They are recommended to be implemented in two phases. Phase one focuses on short-term, low-cost countermeasures, such as regular maintenance and inspection of signing, pavement markings, median modifications, and use of simple AM techniques. Phase two is a long-term, systematic approach for improving geometric design and ITS technologies, education, and enforcement. Transportation agencies should consider the causal factors for the WWD incidents in their jurisdictions and implement countermeasures that address the identified causes. Improvements with traffic signs and pavement markings are cheaper and easier to implement.

3.5.2.1 Short-term, low-cost countermeasures

It was found from field review that besides the geometric features of median openings, a lack of required TCDs can lead to WWD. These types of countermeasures are mainly related to traditional signing and pavement marking by MUTCD. Additionally, as darkness was the common condition for majority of the WWD crashes on divided highways, visibility of TCDs is recommended to be improved by using illuminated signs, adding small led units along the sign's borders to catch a wrong-way driver's attention, affixing red reflective tape to the signposts to enhance nighttime visibility, and adding pavement lights such as reflective raised pavement markers. Older drivers and DUI drivers often contributed to WWD crashes, so enlarging the size of signs, adding a second identical sign and using augmenting warning signing at high frequency WWD crashes locations could draw more attention from drivers. According to Californian Traffic Manual (*THDOT*, *1996*), WW arrows (type V arrows) can be installed on divide highways too. See Table 9 for more general low-cost countermeasures to deter WWD.

3.5.2.2 Long-term, systematic countermeasures

Long-term countermeasures can entail a more comprehensive approach including engineering (TCD maintenances, geometry modification, and ITS applications), enforcement (emergency response, confinement, and radio messages), and education (training). These countermeasures combined can help reduce the likelihood and severity of WWD incidents on divided highways. It is also recommended that field inspections be conducted periodically at high crash segments or median openings of divided highways using the checklist designed in this study. Education strategies can be implemented to improve public awareness and understanding of the basics of road designs and median types, potential WWD risks, what to do when involved in WWD incident, and possible damages to family and society. Education programs should focus especially on older drivers and DUI drivers. Enforcement strategies that could be implemented include adding data-driven DUI checkpoints, stopping wrong-way drivers by using portable spike barriers, and using radio, DMS, or ITS applications to warn right-way drivers of oncoming wrong-way drivers. However, countermeasures alone cannot eliminate WWD incidents by impaired driving, so everyone is encouraged to keep each other from driving while impaired.

CHAPTER 4 EFFECTS OF TURN-PROHIBITION SIGNAL CONTROL AT WRONG-WAY ENTRY POINTS

4.1 Introduction

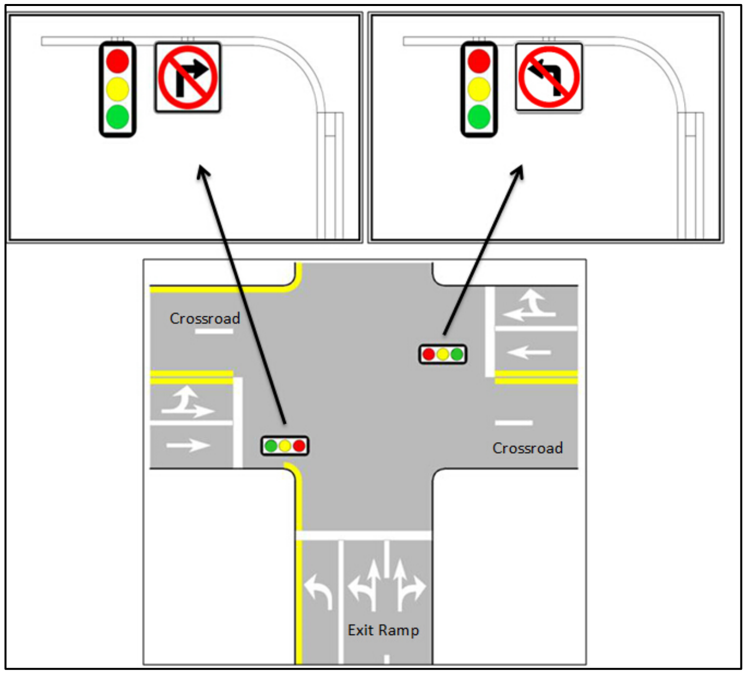
Traffic agencies face challenges in safely accommodating vehicles at the approximately 300,000 signalized intersections in the US (*FHWA, 2013*). Although dedicated straight movement lanes have improved intersection operations, the demand to address wrong-way movements is growing. Wrong-way driving (WWD) refers to movements against the traffic stream on freeways, expressways, interstate highways, and their access ramps. According to the Fatality Analysis Reporting System (FARS), an average of 359 people perished in 269 fatal WWD crashes per year from 2004 to 2011 (*NTSB, 2012*) on US highways. Fatality rate for wrong-way crashes on controlled-access highways is 27 times that of other types of crashes (*NTSB, 2012*). Most WWD crashes on freeways originate from crossroads to exit ramps at interchange terminals (*NTSB, 2012*).

Most past studies on traffic signal indications focused on effects of protected–permissive signal indications, i.e. the FYA, FCY, FCR, and FRA, but few on CG or GA indications. Survey and driver simulator were the common tools for those studies. It was indicated that effectiveness was difficult to measure if the study only focused on exclusive CG and GA lane configurations. From the safety aspect, accidents at CG or GA locations usually result from many different factors. Some drivers may not fully understand unique directional permission by GAs for movement indicated by the arrow only. Few studies have attempted to compare effectiveness between CGs and GAs in terms of preventing WWD.

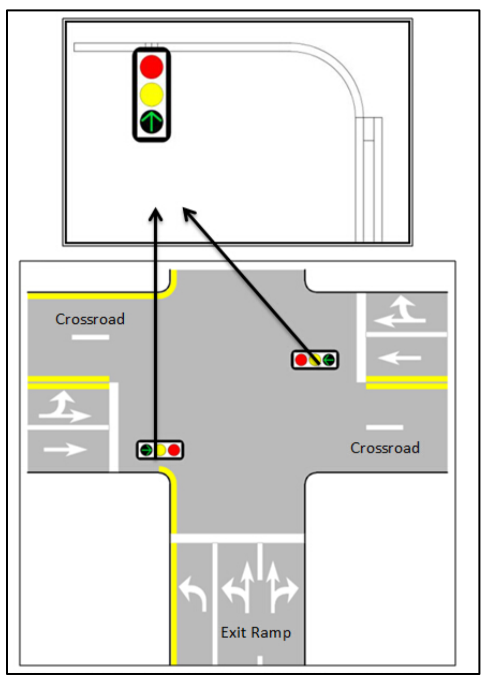
To deter WWD, Green Arrow (GA) or Circular Green (CG) indications are often used with No Right/Left Turn signs at interchange terminals (Figure 20). Per the MUTCD (*2009*), turn-prohibition signs may be post-mounted to supplement traffic signals. Like other traffic signing, the information from a signal is transmitted to the driver within a traffic signal phase through the illumination of a circle-shaped or arrow-shaped indication combined with colors, shapes, orientations, and positions, while complemented via traffic signs. Although the potential benefits associated with traffic indications are known, they may only be achieved when indications are correctly presented to and interpreted by drivers.

Some researchers conjecture that the variety of CG and GA combinations might increase the likelihood of WWD. It is necessary to consider the safety implications of various signal displays and the difficulties in achieving uniformity across states. Factors to consider include driver understanding of the displays and safety effects.

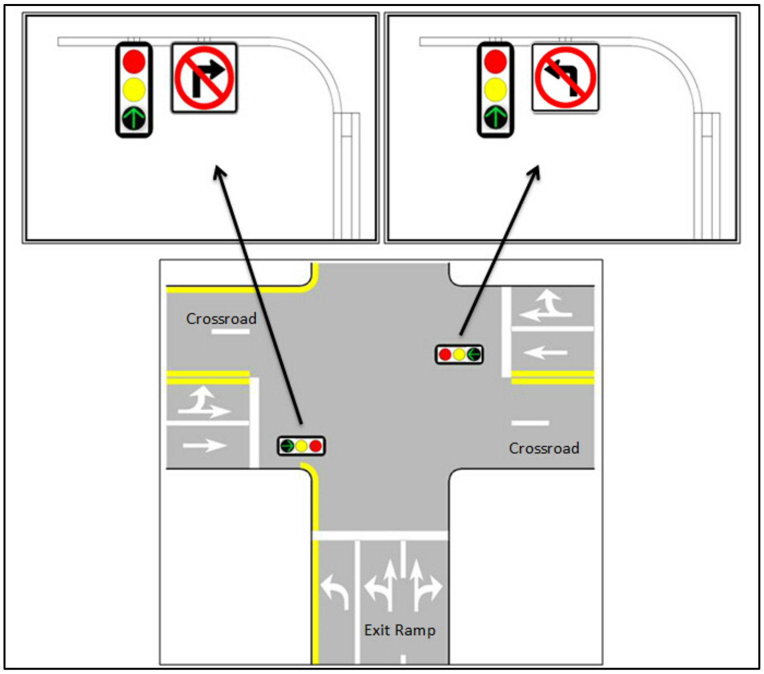
Thus, the effectiveness and application circumstances of different combinations of the two signal indications were evaluated and specified in the study for mitigating WWD at interchange terminals**.** This study performed a comparison of GAs and CGs to deter WWD at interchange terminals through literature review, surveys, and interviews. Particularly, three different combinations of green signals and turn-prohibition signs were discussed on safety advantages and disadvantages: GA + Turn-prohibition Sign, GA Only, and CG + Turn-prohibition Sign.


Crossroad
Crossroad
Exit Ramp

(a)


Crossroad
Crossroad
Exit Ramp

(b)



(c)

FIGURE 20 Illustrations of Circular Greens (a) and Green Arrows (b and c) for turn-prohibition use

4.2 Data Collection

Three surveys were designed and posted online with links distributed through emails. One public driver survey assessed drivers' comprehension and preferences between GA and CG indications. One general WWD traffic agency survey gathered information of GA usage as a WWD countermeasure from agencies involved in WWD research. One comprehensive traffic agency survey obtained traffic agencies' current practices and opinions, regarding GAs and CGs for lane-use indications. Interviews were further conducted for response clarification from participants.

4.2.1 Survey for Drivers

The survey solicited from different ages, genders, races, and education. It contained 10 questions and took 5-10 minutes to complete. It was distributed anonymously in Auburn, Alabama, a college town for the best-case scenario of drivers' comprehension capabilities as the state has lower income and education levels. An information letter served as the cover and documentation of consent, including basic explanations of the research objective, research questions, the recruitment method, and how data would be collected and protected. The first six questions collected participants' demographics, including gender, age, nationality, race, education, and US licensed driver or not. The next three questions tested drivers' comprehension of GAs and CGs by asking if they can make turning movement when facing the three signal displays shown in Figure 21 above, i.e. CGs with turn-prohibition signs, GAs only, and GAs with turn-prohibition signs. The last question inquired drivers' preference among the three displays as a better indication of correct movements when presented.

4.2.2 Surveys of WWD Researchers and Traffic Engineers

4.2.2.1 Survey I

This general questionnaire surveyed representatives during the 2013 National WWD Summit, a group of active WWD researchers and practitioners from the National Transportation Safety Board (NTSB), Federal Highway Administration (FHWA), American Traffic Safety Services Association (ATSSA), Illinois State Toll Highway Authority, state departments of transportation (DOTs), state police and highway patrols, universities, and consulting firms. Therefore, the answers can indicate a notion of the preference for steady green signals as a countermeasure of WWD from expert opinions and past experience. In the survey, one general question about traffic signal use for WWD prevention is "Does your state use GAs as traffic signal indications at the intersection of exit ramps and crossroads instead of CGs to create a better understanding of the correct movement direction?" This question is part of a comprehensive survey at the first National WWD Summit

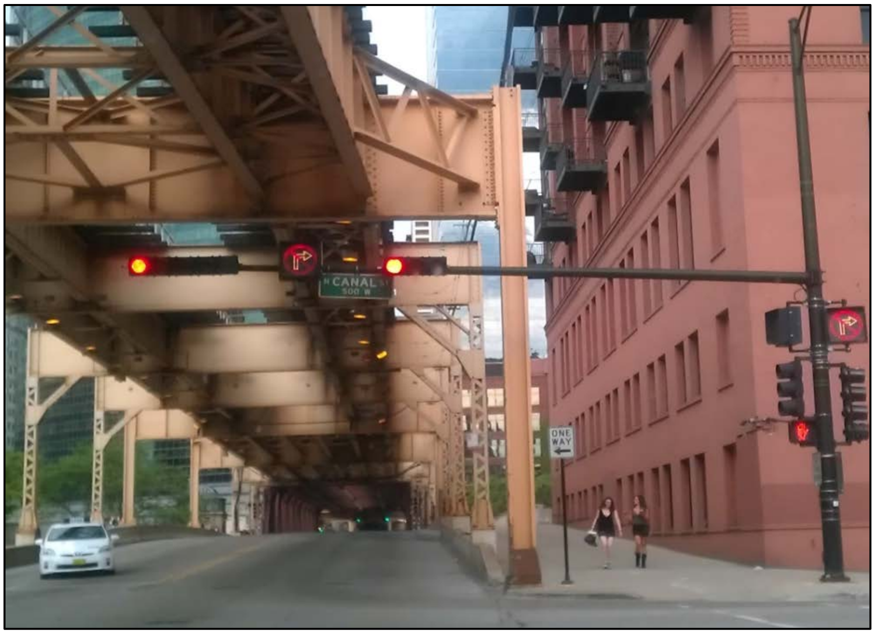
4.2.2.2 Survey II

This comprehensive traffic agency questionnaire surveyed personnel on the National Safety Engineers List and state DOT traffic engineers. The purpose was to collect details about current and emerging practices employed by different traffic agencies regarding the use of GAs versus CGs to reduce WWD at interchange terminals. It contained 15 questions and took 10-15 minutes to complete.

The survey included questions on agencies' application and preferences of steady green signals and turn-prohibition signs for dedicated through lanes regarding size and type, the order and placement manner, installation cost, and application circumstances. Information on the guidelines of GAs and CGs the agency were using was then requested. Opinions of GAs and CGs related causes and suggested countermeasures for WWD at terminals were gathered. In last sections, participants were asked to rank the five combinations of the steady green signals (i.e. GA + turn-prohibition sign (LED), GA + regular turn-prohibition sign, CG + turn-prohibition sign (LED), GA Only, CG + regular turn-prohibition sign, respectively), given five real-world examples (Figure 21) for turn- prohibition use in deterring WWD. They were also asked to rate the overall performances of GA and CG displays for turn-prohibition use considering all possible impact factors, i.e. Equipment Cost, Understandability, Visibility, Current Popularity, Response Complexity, Perception time, Conformity, Easiness of Implementation, and Existing Education Materials.

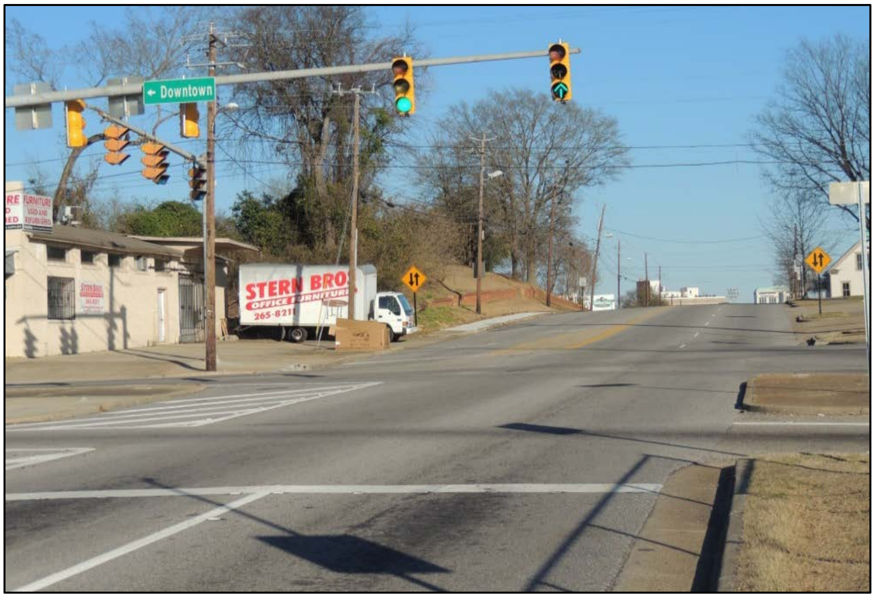
(a) Green Arrow + turn-prohibition sign (LED), Chicago IL

(b) GA + regular turn-prohibition sign, Birmingham AL



Example 1

(c) CG + turn-prohibition sign (LED), Chicago IL



Daytime

Nighttime

(d) GA Only, Montgomery AL

Daytime

Nighttime

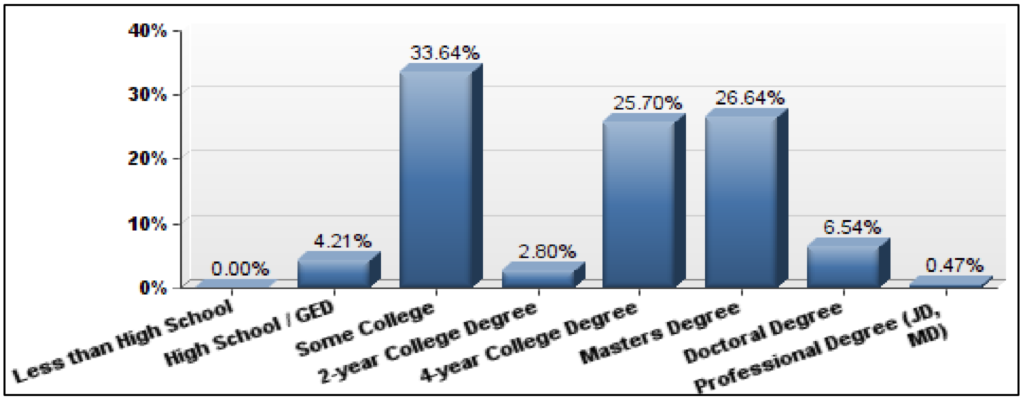
(e) CG + regular turn-prohibition sign, Montgomery AL

FIGURE 21 Real-world examples for steady green displays for turn- prohibition use

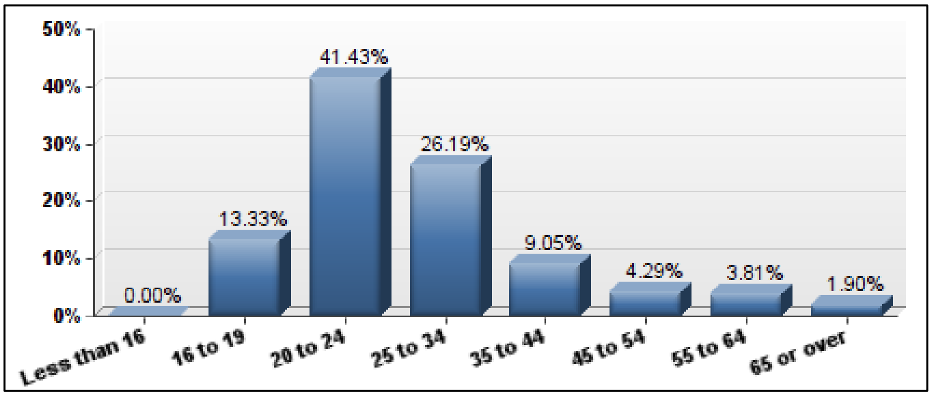
4.3 Data Analysis and Results

4.3.1 Analysis of Public Drivers' Survey

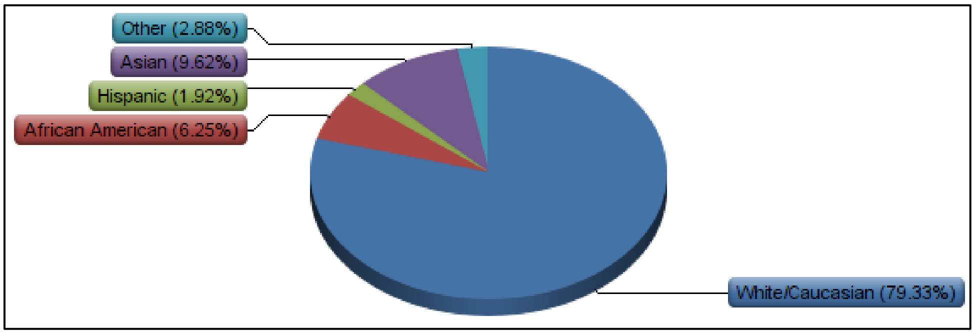
The final sample comprised 273 responses from a demographically well-balanced population of participants. An almost equal number of males (48.56%) and females (51.44%) responded; 98% of participants possessed a US driver's license, an indicator of having required knowledge of driving in the US. Most have a college degree or higher as the survey was distributed among residents in a college town (Figure 22a). Compared with the statistics from the Distribution of Licensed Drivers by Sex and Percentage in Each Age Group in the US (*FHWA, 2014*), participants' age composition distribution (Figure 22b) were consistent with current licensed drivers' age distribution. Figure 22c illustrates race compositions. Regarding the nationality composition of the survey participants, 84% of the participants were US citizens and 16% were not.



(a)



(b)



(c)

FIGURE 22 Education (a), age (b), and race (c) compositions

For indications given by CGs with turn-prohibition signs, 95.63% of participants had no trouble understanding and making the intended movements, while 23.56% did not understand

GAs displayed alone enough to make correct turning decisions. However, when supplemented with turn-prohibition signs, the GA was able to be understood by all (Figure 23).

In the end, the drivers were asked which of the two signals gave them a better indication to keep them from making wrong-way turns at interchange terminals. GAs supplemented by turn-prohibition signs were chosen by 59.96% of all the participant public drivers. CGs with turn-prohibition signs were chosen by 17.79% of the drivers as they expressed that they preferred to see displays of prohibited movements rather than a single traffic signal for permissive moves. Most of them felt that it was hard to see the turn-prohibition signs at nighttime and GAs alone worked better in low-light conditions than CGs with supplemented turn-prohibition signs. 10.26% liked GAs more because they were more visible and forceful. This 10.26% stated the arrows would deter them from making wrong turns due to having less roadway information to digest and more reaction time; 11.99% felt any of the options would work for them (Figure 24).

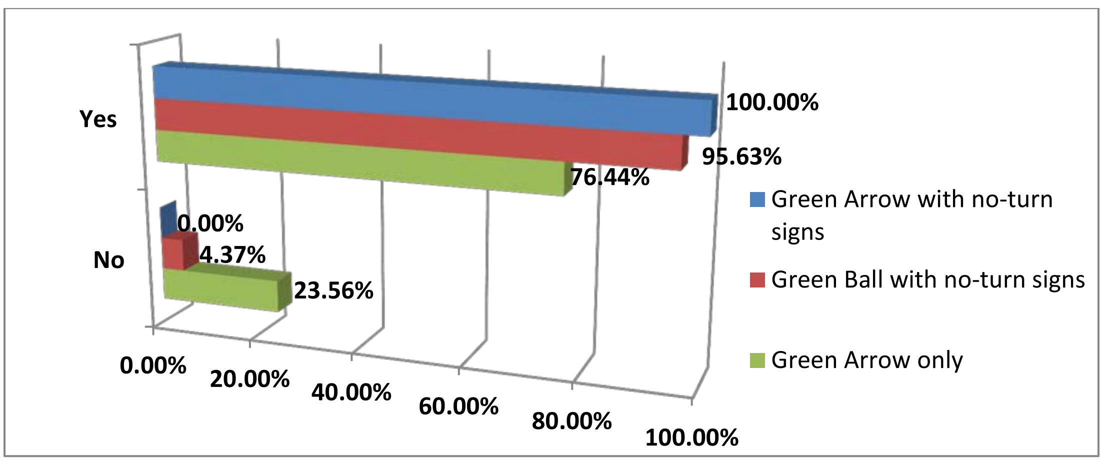


FIGURE 23 Drivers' comprehension

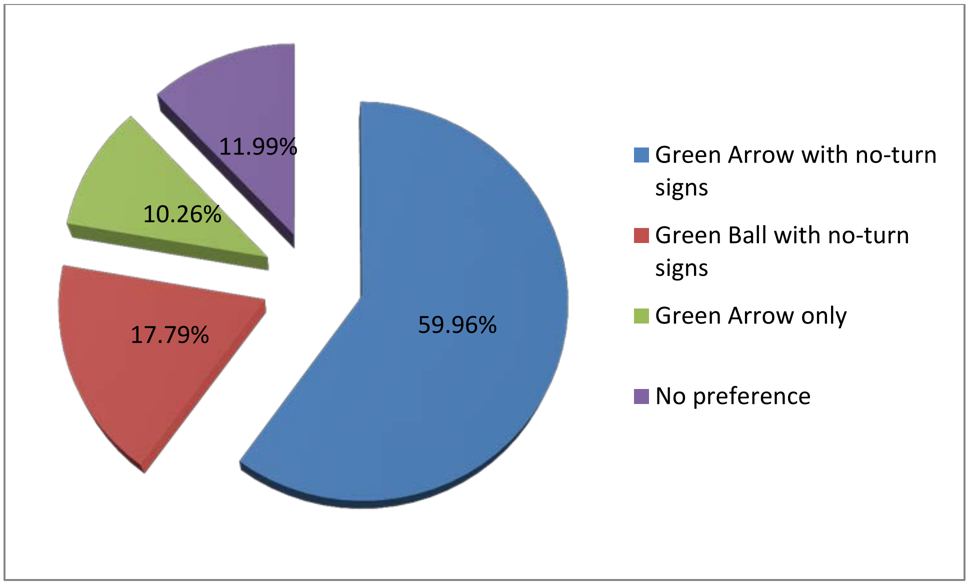


FIGURE 24 Drivers' preferences

Thus, the preference distribution and reasoning behind the choices made by participants were consistent. It showed that GAs gave more visible indications in low-light conditions; if accompanied by turn-prohibition signs, GAs can be better understood by motorists; increasing the visibility of turn-prohibition signs would work better in both well-lit and poorly lit situations for GAs and CGs by, for example, using LED signs. More education about the meaning of GAs among drivers also helps.

Moreover, hypothesis testing was conducted using F-tests, commonly used for deciding whether groupings of data by category are meaningful (*Donaldson, 2014; Tiku, 1971; and Loveland, 2013*) for effects of GAs on enhancing drivers' understanding of turn prohibitions at interchange terminals based on each driver's choice of movements and preference in the survey between GAs and CGs. The null hypothesis was that GAs have no effects on drivers' understanding, denoted by H0; otherwise, Ha: Green Arrows have significant effects. At 95% confidence level, the estimated F-statistic of 0.054 of the coefficient of the independent variable, was smaller than the critical value of $F(\infty)=1.000$. Therefore, the null hypothesis was rejected and GAs have significant effects on enhancing drivers' understanding of turn prohibitions at signalized interchange terminals.

4.3.2 Analysis of Traffic Engineers' Surveys

The first survey was responded by 16 state DOT representatives out of 26 participating states at the first WWD Summit, including Alabama, California, Connecticut, North Carolina, Florida, Georgia, Illinois, Kentucky, Michigan, Missouri, Massachusetts, Mississippi, Oregon, Rhode Island, South Carolina, and Texas. The survey revealed that 38% of participating states have used GAs at interchange terminals to deter WWD. Some representatives mentioned that they were going to implement it statewide to unify traffic signals to reduce drivers' confusion. For instance, the Rhode Island DOT replaced all CGs with GAs at all the interchange terminals in the state in 2014 immediately after the Summit (*Rocchio, 2015*).

In the second survey, respondents included 73 representatives from 12 states: Alabama, Connecticut, Georgia, Iowa, Illinois, Idaho, Massachusetts, Missouri, Michigan, North Carolina, Ohio, and Pennsylvania. It showed 36% of the participating states have installed GAs at interchange terminals; 55% explained applications of GAs in the state's Driver Manual (Figure 25). Participants also noted that today GAs are used just as commonly as CGs. The installation and maintenance costs for the two signals are about the same. It is very cheap and easy to replace CGs with GAs. Additionally, all participants claimed that the MUTCD was the only guideline they followed for the installation and application of traffic signals. When asked for their professional perspectives about the other factors that may influence the effectiveness of GAs and CGs in deterring WWD, all of the surveyed state representatives pointed out that the visibility of supplemental signing and marking has significant influence.

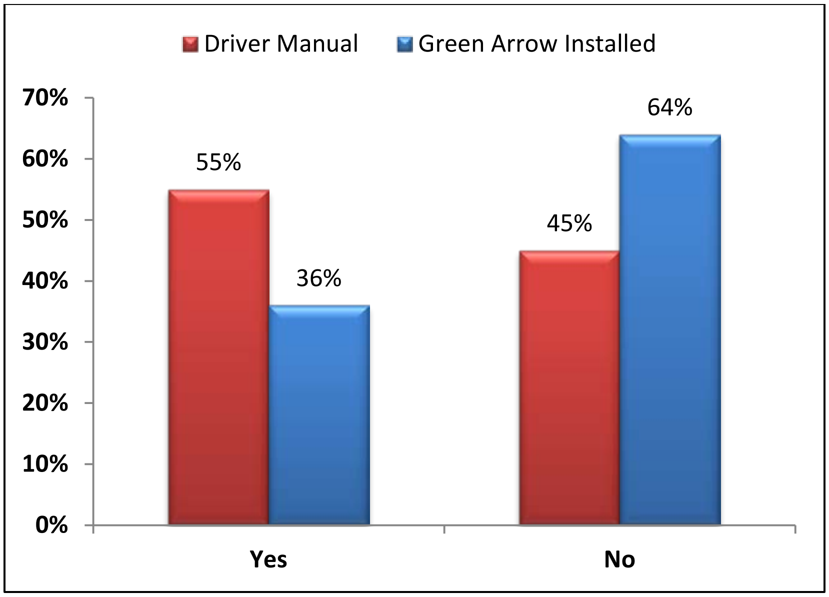


FIGURE 25 Implementation and education

An investigation of supplemental signing indicated that all the surveyed states follow the exact instructions in the MUTCD for placements of No Right/Left Turn at interchange terminals. In particular, the findings indicated that most supplemental signs used at interchange terminals are retroreflective but not self-lit, and a common placement choice (83%) was sized 36 × 36 inches mounted next to signal face on mast arms. rural. However, the participants indicated that most districts in their states either do not or have not used turn-prohibition signs to supplement traffic signals to prevent WWD at interchange terminals; it can be difficult not only for slightly impaired divers but also for the non-impaired to determine the permissive and prohibitive movement at terminals and intersections with one-way streets with current signal displays. The participants believed that drivers' confusion at signalized interchange terminals contributes greatly to this national problem and the countermeasure would improve operation and safety.

For the question of ranking the turn-prohibition use of GAs and CGs based only on WWD prevention effectiveness, the summarized ranking results are: NO.1. "GA + turn-prohibition sign (LED)", NO.2. "GA + turn-prohibition sign (Regular)", NO.3. "CG + turn-prohibition sign (LED)", NO.4. "CG + turn-prohibition sign (Regular)", NO.5. "GA only".

Responses to the last question, to rate the overall performance of GA and CG displays considering all possible impact factors, are summarized in Table 10. Each cell is obtained from the average of all participants' ratings. Scores were given on a scale from 1 to 5 (1= unacceptable; 2= fair; 3= good; 4= very good; 5=excellent). Higher scores indicate better performances. The highest score of each factor is highlighted. It showed that GAs alone perform best in terms of equipment cost and easiness of implementation, but worse in terms of understandability and existing education materials. Nevertheless, when supplemented with turn-prohibition signs, GAs significantly outperformed CGs in terms of Understandability, Visibility, Response Complexity, and Perception Time, which all directly influence driver behavior. LED signs can improve the performance of signals and make GAs achieve the best performance, especially visibility. However, Equipment Cost is a major restriction of its application, which also explains the lower scores of GAs/CGs with LED signs on Current Popularity, Conformity, and Easiness of Implementation.

TABLE 10 Summary of Performance Rating Results

Variable	CG + turn-prohibition sign (LED)	CG + turn-prohibition sign (Regular)	GA only	GA + turn-prohibition sign (LED)	GA + turn-prohibition sign (Regular)
Equipment Cost	2.6	4.0	4.4	**2.6**	4.0
Understandability	3.9	3.4	2.9	4.4	4.1
Visibility	4.3	3.7	3.6	4.8	3.9
Current Popularity	2.4	4.3	2.7	**2.3**	3.1
Response Complexity	3.8	3.4	2.9	4.3	4.3
Perception time	3.9	3.4	3.0	4.5	4.0
Conformity	2.6	3.4	2.6	3.0	3.2
Easiness of Implementation	2.8	4.2	4.3	3.0	4.0
Existing Education Materials	2.6	2.8	2.0	2.0	2.2

Furthermore, by summing up ratings of the variable (Understandability, Visibility, Response Complexity, and Perception time) that are directly affects driver behavior, the overall ratings from the highest to lowest are: 18 for “GA + turn-prohibition sign (LED)”, 16.3 for “GA + turn-prohibition sign (Regular)”, 15.9 for “CG + turn-prohibition sign (LED)”, 13.9 for “CG + turn-prohibition sign (Regular)”, 12.4 for “GA only”. This rating order is consistent with the agencies’ ranking results based only on WWD prevention effectiveness in the previous question, which explains the ranking results.

In addition, according to the phone interviews of traffic engineers from state DOT in Alabama, the current traffic signals in Alabama were initially installed decades ago and have been kept the same since. GAs have been applied to one-way streets in urban areas based on the guidelines at that time for the purpose of helping motorists drive through complicated intersections.

4.4 Conclusions and Recommendations

This study fills in the absence of traffic signal countermeasures to reduce wrong-way incidents at interchange terminals and sets a starting point for further investigations and potential revisions in future editions of the MUTCD. It compared the differences in effectiveness between GAs and CGs on turn-prohibition use based on survey and interview results.

Three surveys were conducted, including one public driver survey that assessed drivers’ comprehension and preferences between GA and CG indications, one general WWD traffic agency survey that gathered information of GA usage as a WWD countermeasure from agencies involved in WWD research, and one comprehensive traffic agency survey which obtained traffic agencies’ current practices and opinions, regarding GAs and CGs for lane-use indications. Interviews were done for response clarification from participants.

The public driver survey results indicate: 96% of drivers have no trouble understanding of the correct movement presented by CGs with turn-prohibition signs; GAs displayed alone may not be fully understood by 24% of drivers, but when supplemented with turn-prohibition signs, driver comprehension achieved 100%. GAs with turn-prohibition signs were preferred by participant drivers to CGs with turn-prohibition signs or GAs alone for prohibited movements rather than a single traffic signal for permissive moves. Furthermore, hypothesis testing indicated GAs have significant impacts on diver behavior.

The general agency survey showed some states involved in WWD research replaced or plan to replace CGs with GAs at interchange terminals to deter WWD. From the other comprehensive agency survey, it is concluded that GAs supplemented with turn-prohibition signs significantly outperformed CGs with same supplemented signs in terms of Understandability, Visibility, Response Complexity, and Perception time. LED signs can improve signal performance and make GA achieve the best performance, especially visibility, but equipment cost restricts its popularity, conformity, and easiness of implementation.

In conclusion, replacing CGs with GAs can help prevent WWD at interchange terminals. Turn-prohibition signs should complement GAs to enhance its performance and driver comprehension. LED turn-prohibition signs can be used with GAs at low-light, under-bridge, and high-crash-risk locations. Recommendation on application and installation circumstances of steady green signals were listed in the order from best to worst based on turn-prohibition and WWD-prevention effectiveness:

1. GA + turn-prohibition sign (LED): applied to complicated high traffic volume intersections, especially in one-way, low-light conditions (e.g., interchange terminals, urban one-way crossroad intersections).

2. GA + regular turn-prohibition sign: applied to complicated and standard intersections, especially for one-way crossroad intersections.
3. CG + turn-prohibition sign (LED) (Turn-prohibition signs shall be mandated, not optional): applied to standard intersections, especially in low-light conditions.
4. CG + regular turn-prohibition sign (Turn-prohibition signs shall be mandated, not optional): applied to standard intersections in good lighting conditions.
5. GA Only: applied to low traffic volume intersections, especially in one-way-crossroad and good lighting conditions.

It should be noted that this ranking is based on survey results and current best practices. It has not been proven by the evaluation of crash data because of the rarity of WWD crashes and the lack of WWD incident records. Further research can be conducted when such data become available.

CHAPTER 5 EFFECTS OF GEOMETRIC ELEMENTS AT WRONG-WAY ENTRY POINTS

5.1 Introduction

Wrong-way driving (WWD) is a major traffic safety hazard at interchange terminals. By definition, WWD comprises two main elements: 1) driving movements against the main direction of traffic flow along high-speed, physically-divided highways (i.e., freeways, expressways, and interstate highways), and 2) accidentally entering a freeway system in the wrong direction from the access on and off ramps of the highways (i.e., executing a U-turn on the freeway mainline, crossing the median through an emergency turnaround, or entering from an exit ramp) (*Zhou et al., 2012;, and ATSSA, 2014*). Past studies suggested partial cloverleaf (parclo) interchanges are highly susceptible to WWD movements, particularly when exit ramps and entrance ramps are closely spaced together due to their geometric features (*Morena and Leix, 2012*).

Geometric elements refer to the branch of highway engineering concerned with the positioning of the physical elements of roadways according to standards and constraints. The basic objectives in geometric design are to optimize efficiency and safety while minimizing cost and environmental damage (*ASSHTO, 2011*). Figure 26 illustrates general geometric elements at parclo interchange terminals, including median type, median width, turning radius, and channelized island.

This study identifies possible geometric elements to direct driver movements at parclo interchange terminals and quantifies relationships between these geometric elements and WWD based on crash and field data analysis to help drivers' decision making and movements, which ultimately, improves traffic safety. Accordingly, this study provides guidelines for parclo interchange terminal design and countermeasures for WWD prevention.

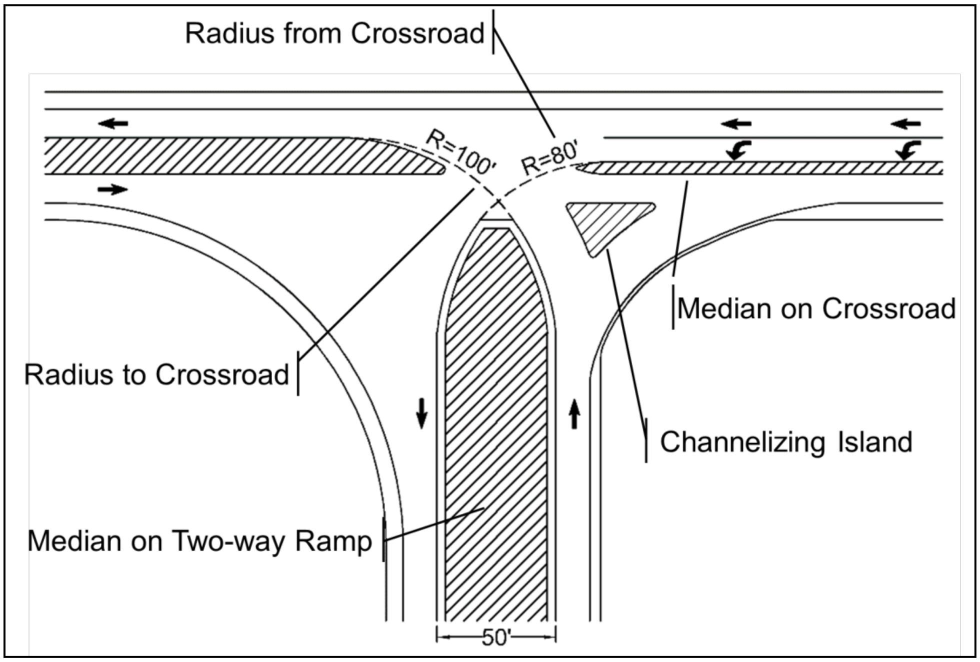


FIGURE 26 Geometric elements at parclo interchange terminals

5.2 Data Collection

5.2.1 Study Sites

Forty-four signalized ramp parclo interchange terminals in Illinois were identified as study locations based on high-resolution aerial photography, street views, and the GIS capabilities of the Google Earth Professional software. The coordinates of each study location were recorded. All left-turning angles from crossroads to entrance ramps were verified to be right angles. Measurements were conducted with measuring tools of Google Earth for each study location and verified in the field, including the turning radius for left-turn drivers on crossroad, median type on crossroad, and the median width between exit ramps and entrance ramps. In addition, two lengths were measured, i.e. $L_{1=}$ the length from the stop line of the left-turn lane(s) to the centerline of the median on two-way ramps and L=the length of the entire intersection (the distance between the stop lines in two opposite directions on the crossroad). The stop line positioning through the intersection was calculated by the following equation (2):

$$S\ (\%) = \frac{L_1}{L} * 100 \qquad (2)$$

5.2.2 WWD Crash Data

For WWD crashes in the selected sites, one crash database for Illinois of a six-year period (2004-2009) was obtained from the Highway Safety Information System (HSIS) of the FHWA which included crash coordinates and wrong-way movement descriptions. Firstly, the route number, roadway description, and roadway functional class of crashes were used to verify the crashes were at interchange vicinities, with street views and aerial photography of Google Earth used as supplementary tools. Then, crash narratives were reviewed to confirm that each crash was resulted from WWD and to determine whether the wrong-way entries occurred at interchange terminals. Thus, wrong-way entry points for each crash were identified or estimated.

Based on all hardcopy crash reports review, 217 WWD crashes were identified in total in the six-year period in Illinois, fifteen of which occurred at signalized ramp terminals of parclo interchanges, including confirmed and possible WWD entries.

5.3 Data Analysis

The effects of the existing applications of geometric elements were analyzed for the 44 study sites. The frequency of WWD crashes experienced at study sites was calculated for each technique. The most common condition for WWD entries for each technique is stop line positionings on crossroad more than 60%, turning radii larger than 100ft., the traversable median, and medians 10 ft. or less (See Table 11 column 3).

Furthermore, the Peto odds ratio (OR) assessed the contribution of several geometric elements and WWD using a 95% confidence interval as the relative measure at statistical significance level of 0.05. This calculation is based on the null hypothesis that treatment has no effect on outcome. Under the null hypothesis, the difference between the observed and the expected would have zero difference and variance. The advantage of the OR method over direct calculations is that it allows for zero results without generating infinity, which is suitable for some categories of variables in the dataset that have no events. The method has been used before for crashes analysis (*Wang, 2008* and *Ponnaluri, 2016*). The OR value reflects the impact of a specific category with larger numbers reflecting greater contribution. The results were presented in Table 11. It reveals that WWD occurrences increase when the stop line positioning on crossroad is more than 60%, the turning radius is larger than 80ft., the median is traversable, and the medians width is within 20-30 ft. between ramps. At statistical significance level of 0.05 (depicted as red text in the table), significant differences exist between different categories of each variable.

TABLE 11. Analysis Results of Geometric Elements vs. WWD Crashes and ORs

Variable	Category	No. of WWD Crashes	WWD Crash Frequency (%)	Odds Ratio
Stop line Positioning on Crossroads	30%-40%	2	13.33	1 (Reference)
	40%-50%	1	6.67	0.24
	50%-60%	3	20.00	0.45
	More than 60%	**9**	**60.00**	5.24
Turning Radius from Crossroads	50 ft. and less	1	6.67	1 (Reference)
	51 to 60 ft.	0	0.00	0.29
	61 to 70 ft.	0	0.00	0.22
	71 to 80 ft.	0	0.00	0.11
	81 to 90 ft.	2	13.33	2.54
	91 to 100 ft.	3	20.00	3.53
	More than 100 ft.	**9**	**60.00**	1.79
Type of Median on Crossroads	Non-traversable	5	33.33	1 (Reference)
	Traversable	**10**	**66.67**	1.39
Median Width between Exit and Entrance Ramps	10 ft. and less	**5**	**33.33**	1 (Reference)
	10 to 20 ft.	4	26.67	0.17
	21 to 30 ft.	3	20.00	3.85
	31 to 40 ft.	3	20.00	3.11
	41 to 50 ft.	0	0.00	0.49
	51 to 60 ft.	0	0.00	0.15
	More than 60 ft.	0	0.00	NA

Note: “NA” means no interchange terminal is with median widths on ramps more than 60 ft.

5.4 Conclusions and Recommendations

This chapter summarizes the existing applications of geometric elements that have been proposed to deter wrong-way movement at parclo interchange terminals. It analyzed the collected WWD crash and geometric design data to quantify the relationships between geometric elements and WWD at parclo interchange terminals. The most common conditions for WWD entries for each technique are stop line positionings on crossroad more than 60%, turning radii larger than 100ft., the traversable median, and medians 10 ft. or less. The highest OR belongs to the stop line positioning on crossroads, indicating high importance of this factor. Turning radii larger than 80 ft. from crossroads to two-way ramps are twice or more susceptible to WWD entries. Traversable median are more prone to WWD entries. Median widths more than 40 ft. between ramps are less vulnerable to WWD entries.

This study establishes the foundation for countermeasure developments and design guide modifications to improve safety at parclo interchange terminals, including stop line positioning no more than 60%, Turning Radii less than 80 ft., using non-traversable median, and median widths less than 40 ft. Although the methodology identified vulnerable geometric designs to WWD and validated the differences between geometric elements with field and crash data, it cannot determine what caused the differences of driver behavior. Further study on driver behavior can be explored based on this study.

CHAPTER 6 EFFECTS OF INTERSECTION BALANCE AT WRONG-WAY ENTRY POINTS

6.1 Introduction

The Freeway interchange terminals are essential components in the highway system. As the access points to freeways, interchange terminals are the main source of accidents and congestion. Several previous studies have found that a majority of freeway crashes occur at interchange terminals, and inappropriate terminal design has caused server safety problems like wrong-way driving (WWD) crashes. WWD has become a major traffic safety hazard at interchange terminals, especially when exit ramps and entrance ramps are closely spaced. Providing drivers with open sight distance on entrance ramps can significantly reduce wrong-way movements. An adequate sight distance not only provides drivers on crossroads with a better view of ramp terminals but also helps drivers distinguish between entrance and exit ramps at night via headlights.

One essential design element in the planning, design, and operation of interchange terminals is a proper intersection balance that can provide a safer and smother movement of left-turn vehicles while maintaining vehicular throughput and access to adjoining ramps. It needs to reflect sound traffic engineering principles and driver behavior. An appropriate intersection balance throughout the entire width of the intersection has decisive effects on the sight distance at intersections. At the intersection of two-way ramps and crossroads for a partial cloverleaf (parclo) interchange, a desirable throat width can be achieved by moving stop lines for left turns from the crossroad forward so that motorists have a better view of the entrance ramp and a better turning radius. The WSDOT (*2013*) proposed that the distance between the stop line for left turns from a crossroad and the middle of the median separating on and off ramps be no more than 60%

of the entire intersection width (see Figure 27). In this study, intersection balance is defined as the percentage which the left-turning point is of the way through the intersection.

Despite the importance of intersection balance, it is often overlooked in current roadway design and site planning efforts. Driver behavior is one crucial element to consider in the design of signalized intersections, because driver behavior was identified as a critical contributing factor to about 96% of intersection-related crashes (*Tijerina et al., 1994*). Therefore, besides crash analysis, the intersection balance study should also focus on the element of time and its relationship to the driving task, which includes perception, reaction, navigation, and execution of the necessary maneuvers. However, research on different intersection balances on WWD and driver sight distance have not been performed because of the lack of driver behavior data of good quality.

Thus, this study evaluated the effects of intersection balances on WWD, intersection sight distance, and driver behavior. Such work may be beneficial for improving freeway traffic safety and operational performance through controlling access location by proper intersection balance design using the second Strategic Highway Research Program's (SHRP 2) Naturalistic Driving Study (NDS) dataset. It also provides rationale for using the naturalistic data to quantify relationships between human and environmental factors for better roadway design, which are unknown from standard crash analysis. Accordingly, recommendation and validation on optimal intersection balance design at parclo interchange terminals can be made.

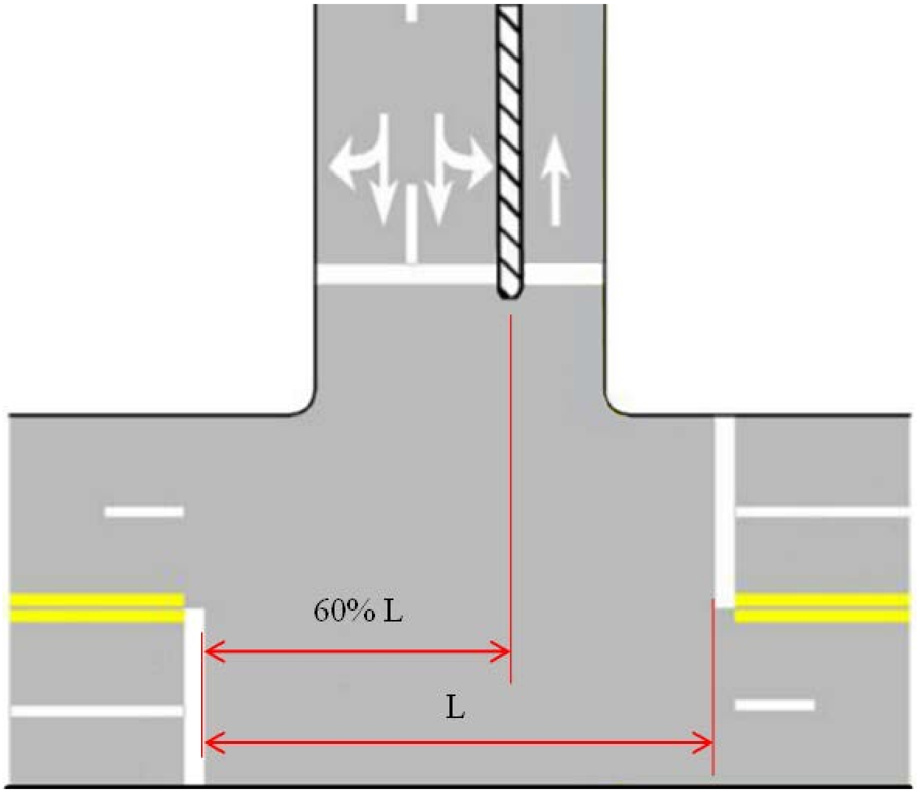


FIGURE 27 Current best practice of the parclo interchange terminal design

Guardrails or concrete median barriers are often used to separate adjacent entrance and exit ramps at parclo interchanges. Past studies (*Bao et al., 2015*) have shown that guardrails may block left-turn drivers' view of entrance ramps when they extend to the stop line of the exit ramp. Drivers making a left turn onto a freeway need to see the delineation of entrance ramps, such as pavement markings, curbs, and other elements, to detect the correct path. Although median barriers used on two-way ramps can prevent catastrophic median crossover collisions and help direct vehicles onto the correct roadway, they can also create sight obstacles and increase the likelihood of WWD if they are overly long. Figure 28 shows an example of a median barrier (guardrail) blocking drivers' view of the throat of an entrance ramp in Michigan. Thus, it is important to properly design the median barrier far enough from the terminal so that a left-turning driver can recognize the entrance ramp.

FIGURE 28 Median barrier blocking driver view of the entrance ramp

The objective of this study was to evaluate the effects of intersection balance on WWD, intersection sight distance, and driver behavior, and to develop new general guidelines for median barrier design to improve drivers' view of entrance ramps at signalized ramp terminals of parclo interchanges.

6.2 Data Collection

6.2.1 Study Interchanges

Forty-four signalized ramp terminals of parclo interchanges in Illinois were identified as study locations based on high-resolution aerial photography, street views, and the GIS capabilities of the Google Earth Professional software. The coordinates of each study location were recorded. All left-turning angles from crossroads to entrance ramps were verified to be right angles. Using the line, path, and ruler functions of Google Earth, two lengths were measured, that is, $L_{1=}$ the length from the stop line of the left-turn lane(s) to the centerline of the median on two-way ramps and L=the length of the entire intersection (the distance between the stop lines in two opposite directions on the crossroad), as shown in Figure 27. The stop line positioning through the intersection was calculated by the equation (2).

6.2.2 WWD Crash Data

For WWD crashes at the selected sites, one crash database for a ten-year period (2004-2013) was obtained from the Highway Safety Information System (HSIS) of the FHWA including crash coordinates and wrong-way movement descriptions. Accordingly, wrong-way entry points for each crash were estimated. The route number, road description, and roadway functional class for the crashes were used to determine whether the wrong-way entries occurred on interchange terminals. Street views and aerial photography were used as supplementary tools to verify crash locations and entry points.

After reviewing all hardcopy crash reports, eighteen WWD crashes were identified at 44 selected signalized ramp terminals of parclo interchanges. Twenty-one ramp terminals were identified as wrong-way entry points (one confirmed, ten first possible, eleven second possible, and one third possible) based on the narratives in crash reports. One crash may have more than one estimated WWD entry point. One parclo interchange may have more than one ramp terminals. One ramp terminal may experience more than one WWD entry (In the study, two ramp terminals each experienced two WWD crashes). So the number of WWD entry points is not equal to the number of crashes.

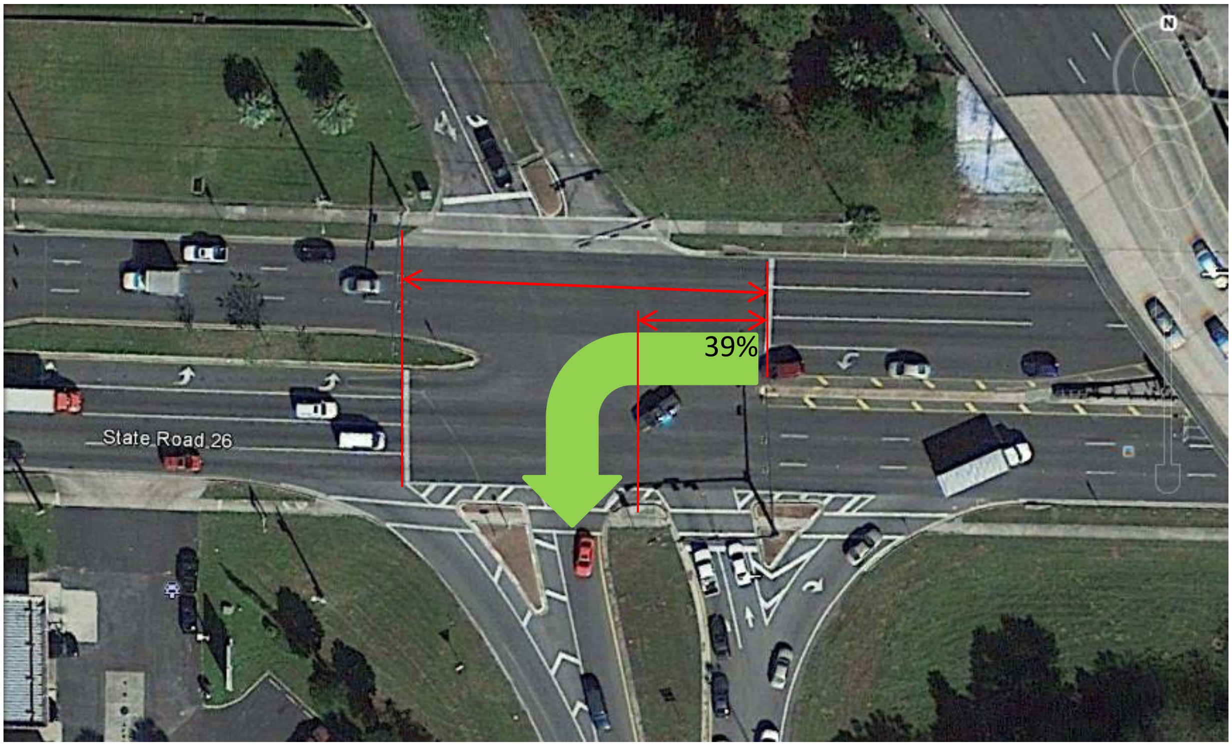
6.2.3 NDS Study Sites

The study locations were selected from the Roadway Information Database (RID) (*CTRE, 2017*), a database containing 1) Roads most frequently driven by NDS participants and 2) Roads of greatest interest to safety researchers. Table 12 includes three signalized terminals of parclo interchanges with different intersection balances (<50%, 50%-60%, and >60%). The

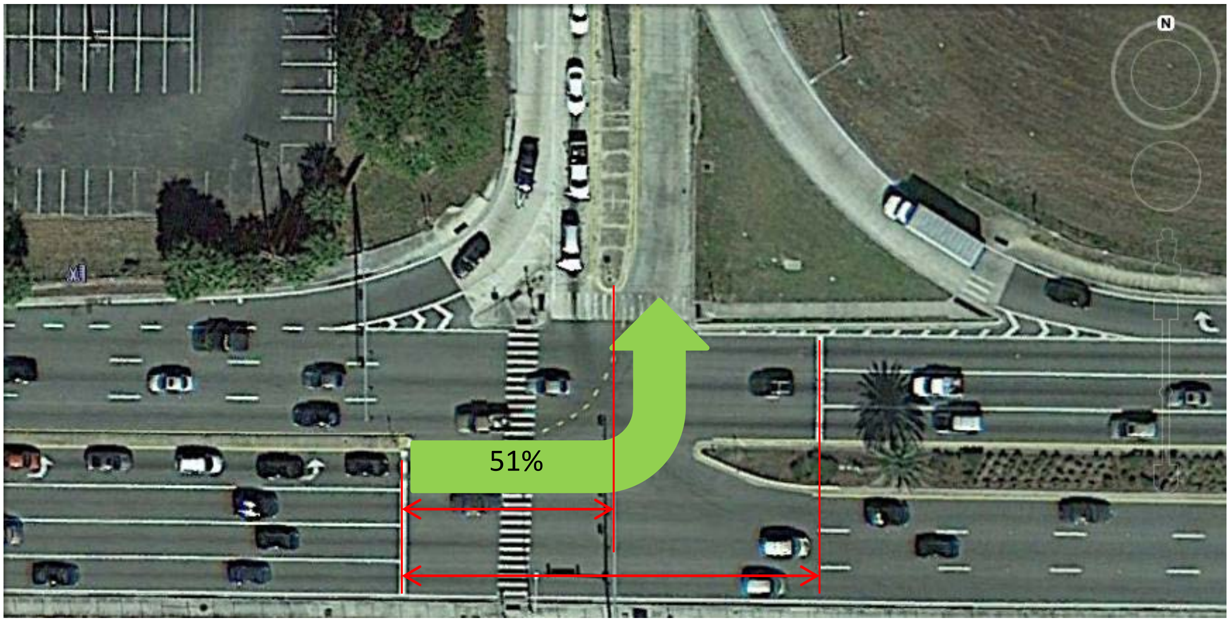
Street View of each location were shown in Figure 29, based on high-resolution aerial photography, street views, and the GIS capabilities of the Google Earth Professional software. In addition, two lengths were measured, i.e. L1: the length from the stop line of the left-turn lane(s) to the centerline of the median on two-way ramps and L: the length of the entire intersection (the distance between the stop lines in two opposite directions on the crossroad). The intersection balance of each interchange terminal was calculated by Equation (2). LINK IDs were identified for the approaches of interest at each location for NDS data request and for linkage between NDS data and RID data. Figure 29 illustrates the locations and vehicle movements of interest (indicated by arrows). A manual review was subsequently conducted of each route ID using Google Earth in order to verify whether the segment was actually at the selected interchange terminal.

TABLE 12 Selected Signalized Terminals of Parclo Interchanges

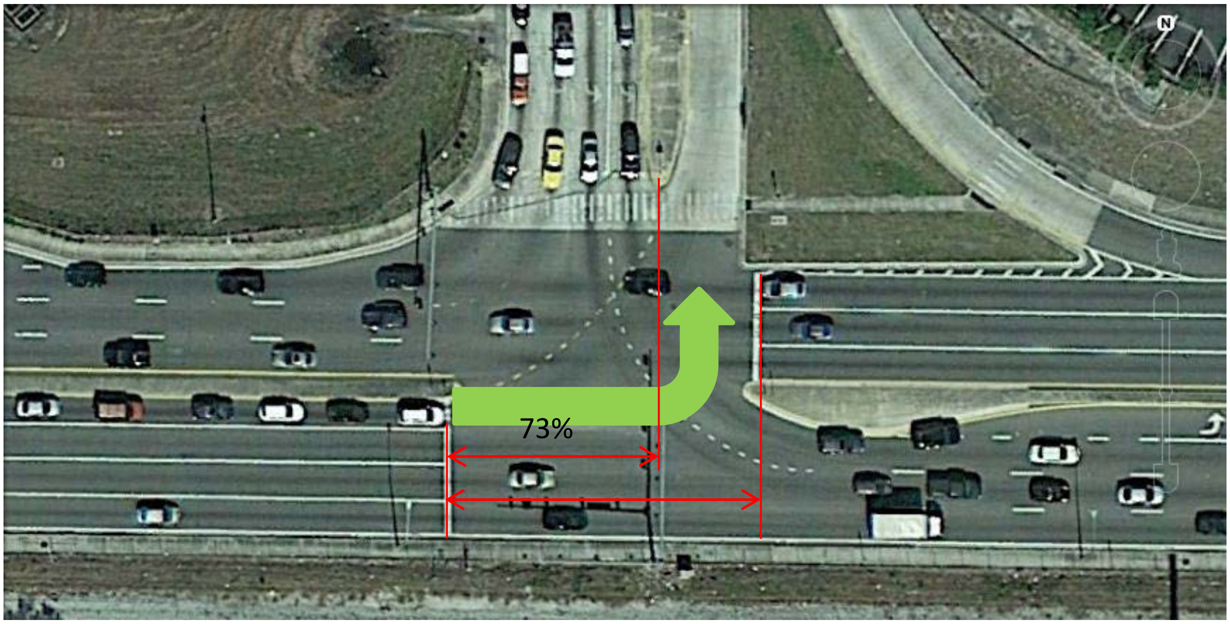
Location	Intersection Balance	Interchange	Interchange Type	Turning Angle	State
Location 1	38.78%	I-75/State Road 26	Parclo	90 degree	FL
Location 2	51.43%	I-275/E Busch Blvd	Parclo	90 degree	FL
Location 3	73.33%	I-275/E Busch Blvd	Parclo	90 degree	FL



(a) Location 1 with marked approaches of interest



(b) Location 2 with marked approaches of interest



(c) Location 3 with marked approaches of interest

FIGURE 29 Selected signalized terminals of parclo interchanges

6.2.4 NDS Data

This research leverages the data from the SHRP 2 NDS database (*VTTI, 2017*) to provide a framework for better understanding the relationships between driver behavior and intersection balance. For the purposes of this proof-of-concept study, data were collected at two levels of detail. First, information was obtained as to the general characteristics associated with each trip from the SHRP 2 NDS InSight website. Subsequently, more detailed information are requested from the SHRP 2 NDS database by the Virginia Tech Transportation Institute (VTTI), e.g., speed, acceleration/deceleration, and GPS location information, as well as the RID, e.g., roadway geometry, traffic control, speed limit, etc.

The NDS dataset includes data from participants, vehicles, and trips. The required parameters to collect for each category were determined before the data request, including data of Videos by all cameras during the trips of interest (Forward View and Right-Rear View), Drivers (Driver Physical and Psychological State, Driver Risk Perception Scale, and Driver

Demographics and Driving History), Vehicles (Vehicle Type, Vehicle Age and Condition, Vehicle Technologies and Equipment), and Trips (Time Series of Trips which are of a time-series nature, providing details of the geographic location, speed, acceleration/deceleration, deceleration, turn rate, brake pedal activations, of each trip). The trip data were comprised of a series of comma-separated values (CSV) files (one file for each event each file includes GPS location information at 1.0-s intervals, as well as speed and acceleration/deceleration data at 0.1-s intervals).

The number of trips and participant data per intersection were specified: for the selected locations with a trip density of less than or equal to 250 trips, all the trips were requested. For locations with a trip density of larger than 250 trips, 250 trips were requested, of which trips driven by different drivers and trips containing “Events (defined by SHRP 2 NDS)” were preferred.

Thus, information on a total of 545 trips by 137 participant drivers was available and obtained from the NDS dataset. These trips with the traversals of interest were requested from the trip pool passing though the predefined 3 locations. 21 trips by 12 drivers were at Location 1. 496 trips by 111 drivers were at Location 2. 28 trips by 14 drivers were at Location 3. None of the trip data include crash or near-crash events, because data were unavailable for the trips with crash or near-crash events, which were primarily comprised of specifics where the associated GPS data could have potentially allowed for the identification of specific drivers and crash/near-crash locations.

6.3 Data Reduction

The collected NDS trip data include front and rear videos, sensor data (speed, acceleration/deceleration, etc.), and supplementary data (driver risk perception based on questionnaires, driver driving history based on questionnaires, vehicle information, etc.). The researchers reviewed all these data and retrieved pre-defined events. Video review was conducted for each trip to verify they have occurred on the study sites and included the maneuver of interest, as well as to extract the portion of left-turn maneuver between the stop line and the entrance ramps out of each event. Cases with missing data were removed. The retrieved data then were validated by conducting data type checks, image checks, consistency checks, range checks, and format checks. After validation, the validated events were compiled further into the 3 study sites for analysis. The number of events at each study site is different. To apply a cross-section comparison method, an equal number of events were extracted from the requested data for each study site. Thus, after data reduction, data of 11 events by passenger cars were selected for each study sites for analysis.

6.4 Data Analysis

This section includes three parts: analysis of stop line positioning data, WWD crash data, and the median barrier length on two-way ramps. Details can be found in the following paragraphs.

6.4.1 Analysis of Stop Line Positioning Data

The stop line positioning data for the 44 signalized ramp terminals of parclo interchanges in Illinois were collected and analyzed, including 6 ramp terminals with stop line positioning of 30%-40%, 10 with stop line positioning of 40%-50%, 16 terminals with stop line positioning of 50%-60%, 9 with stop line positioning of 60%-70%, and 3 terminals with stop line positioning of 70%-80%. None were under 30% or above 80%.

Figure 30 illustrates the density of the number of interchange terminals in each interval of stop line positionings. The frequency fits a logistic distribution as the red line showed at a 95%

confidence level. The hypothesis was tested by the Chi-square test and the Kolmogorov-Smirnov goodness-of-fit test (*Kirkman, 1996; Hazewinkel, 2001; Glover et al., 2008; and Laub and Kuhl, 2015*), where the null hypothesis was H_0: The sample follows a logistic distribution, otherwise H_a. As the computed p-values were greater than the significance level alpha=0.05, the null hypothesis H_0 cannot be rejected. Conclusively, the stop line positioning follows a logistic distribution. Thus, the odds ratio for the stop line positioning of 50%-60% for parclo interchange terminals is the highest, while it decreases with stop line positionings smaller and larger than 50%-60%. The probability of the number of parclo interchanges in 50%-60% increasing has less effect on stop line positionings that are very low or very high, and a much larger effect for the stop line positioning in the middle. For example, if the odds of a stop line positioning in 30%-35% is 1 to 2, the odds of a stop line positioning in 50%-55% is about 10 times as large, or about 10 to 20, while the odds of a stop line positioning in 70%-75% is about equal. In other words, for every 20 stop line positioning in 50%-60% in a sample of parclo interchange terminals, 10 might exist in the same sample.

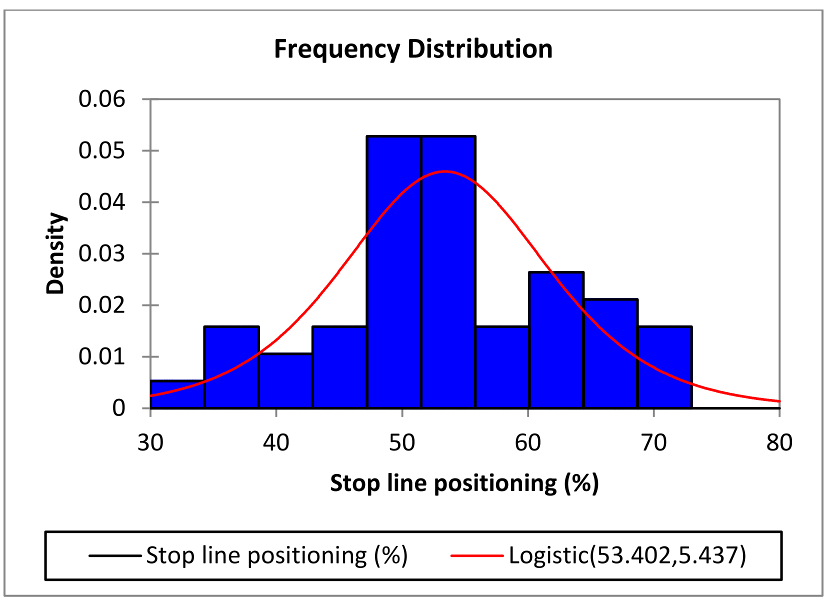


FIGURE 30 Frequency distribution of interchange terminals vs. stop line positionings

The analysis showed that although there is no guideline in current MUTCD indicating the appropriate design of the stop line positioning, the stop line positioning of 50%-60% was frequently used by state departments of transportation (DOTs) and accounted for a large proportion of existing ramp terminals at parclo interchanges.

6.4.2 Analysis of WWD Crash Data

To evaluate the effects of stop line positioning on WWD crashes, the following method was developed. First, the percentage of WWD crashes and the percentage of the type of interchange terminals where WWD crashes have occurred in each category of stop line positioning for each type of stop line positioning were calculated separately. The percentage of WWD crashes for each type of stop line positioning was calculated by the frequency of WWD crashes for this type of stop line positioning in the study period divided by the total number of WWD crashes for all different types of stop line positioning. See equation (3). The percentage of the type of interchange terminals where WWD crashes have occurred in each category of stop line positioning was calculated by the number of that type of interchange terminals divided by the number of all types of interchange terminals in total. See equation (4). By dividing the percentage of WWD crashes by the percentage of that type of interchange terminals, the ratio of the percentage of WWD crashes versus the percentage of interchange terminals was determined for each type of stop line positioning. See equation (5).

$$P_c = n/N * 100\% \qquad (3)$$

$$P_i = i/I * 100\% \qquad (4)$$

$$R = P_c/P_i \qquad (5)$$

Where:

n: the frequency of WWD crashes for this type of stop line positioning in the study period

N: the total number of WWD crashes for all different types of stop line positioning

P_{c}: the percentage of WWD crashes for one type of stop line positioning

i: the number of that type of interchange terminals

I: the number of all types of interchange terminals in total

P_i: the percentage of the type of interchange terminals

R: the ratio of the percentage of WWD crashes versus the percentage of interchange terminals

Figure 31 demonstrates the ratio between the percentage of WWD crashes and the percentage of that type of interchange terminals where WWD crashes have occurred. It suggests that the lowest percentage of WWD crashes was at interchange terminals with the stop lines located 40%-50% through the intersection. Moreover, when the stop line is located more than 60% through the intersection, the percentage of WWD crashes increased significantly. The WWD crash data analysis suggests that the current guideline from WSDOT, to locate the stop line at no more than 60% of the way through the intersection, can effectively mitigate WWD crashes.

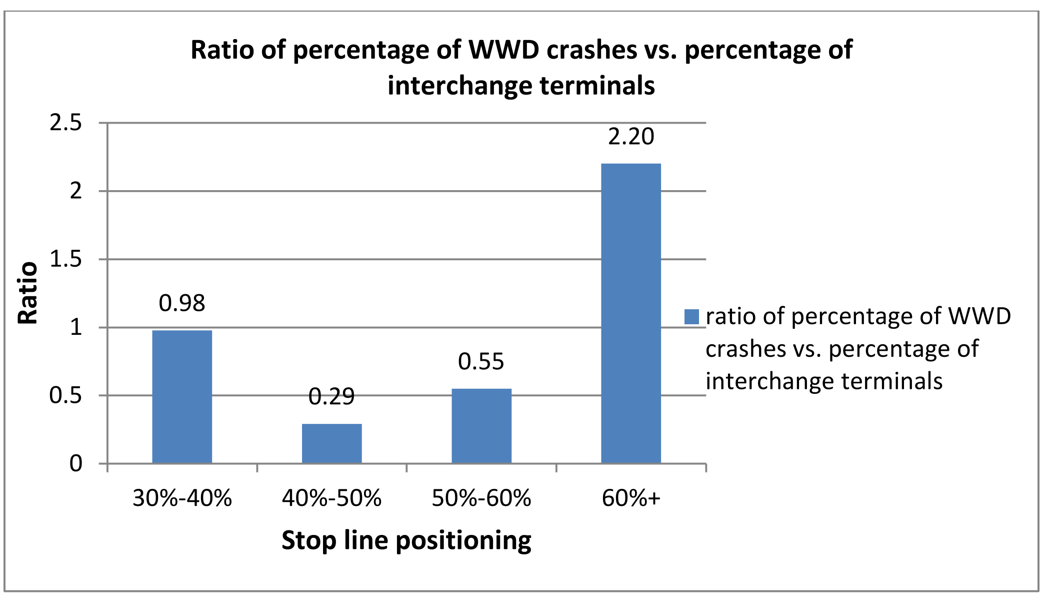


FIGURE 31 Ratio of percentage of WWD crashes vs. percentage of interchange terminals

6.4.3 Analysis of the Median Barrier Length on Two-Way Ramps

A WWD incident, despite faded, damaged, or misplaced roadway signing and marking, is likely to result from a driver misreading environmental information. Especially when it is hard to identify signing and marking in low light conditions at nighttime, people rely mainly on environmental information to make movements. Therefore, an analysis of how drivers process roadway environment information to determine the right way from the wrong way can exhibit drivers' ability to interpret the roadway environment and provide a solid basis for the geometry design of roadways. To bridge the gap, a 3D analytical model of roadway environments was developed from a driver's perspective of the intersection sight distance, where vertical and horizontal curves of different approaches overlap at the intersection of 3D highway alignments.

Based on the perspective and simulation techniques from traditional architecture and landscape design, the 3D model considered human cognition to reveal the physical characteristics of the highway alignment and the driver's and passengers' visual and psychological comfort. For the worst-case scenario for drivers' sight distance, a passenger car was chosen as the experimental vehicle, where the driver's viewing angle was the lowest and the available sight distance was the shortest. A driver is the receiver of roadway environmental information with different view heights and sights. The vehicle body and rear mirrors may obstruct the line of sight for drivers with an acute-angle approach to their left. Although drivers can freely adjust their necks and eyes, their head is relatively fixed regarding the driver's seat. Therefore, to analyze the external roadway environment information, the driver's eye position was assumed at 25 cm (approximately 10 inches) above the center of the seat backrest denoted by a 3D cone; the driver's effective sight was constrained by the car window frames denoted by a 3D rectangular plane (see Figure 32).

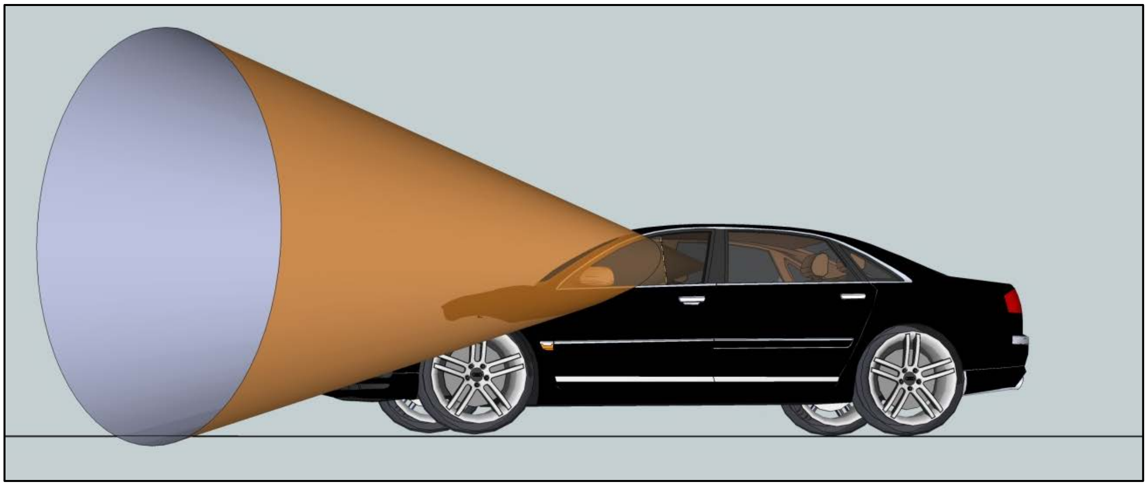


FIGURE 32 Illustration of driver's effective sight

Median barriers or guardrails, if extended too far, can block left-turn drivers' view of the entrance ramp terminal on the crossroad and increase the possibility of making a wrong left turn to enter the exit ramp. Thus, the effects of median barrier length at terminals with different stop line positioning on drivers' perception of right-way information were evaluated. Four 3D simulation models were developed to determine the appropriate position for median barriers on two-way ramps for different stop line positions at signalized ramp terminals of parclo interchanges. 3D roadway environments were created to simulate certain combinations of the stop line positioning and the median barrier length at a parclo interchange terminal.

Hence, the driver's view of the roadway environment information was detailed, reflected, and extracted according to the theory of perspective imaging. For example, in Figure 33, the right way, where the intended movements were, was highlighted in each model as the circled red-colored area among the other identifiable roadway information in the driver's perspective. The percentage of the right-way information was calculated for each 3D model to quantify the amount of roadway information using image analysis techniques. The higher percentage of the right-way information collected by the driver's eyes, the more likely the driver is to find the right way and make the correct turning movements.

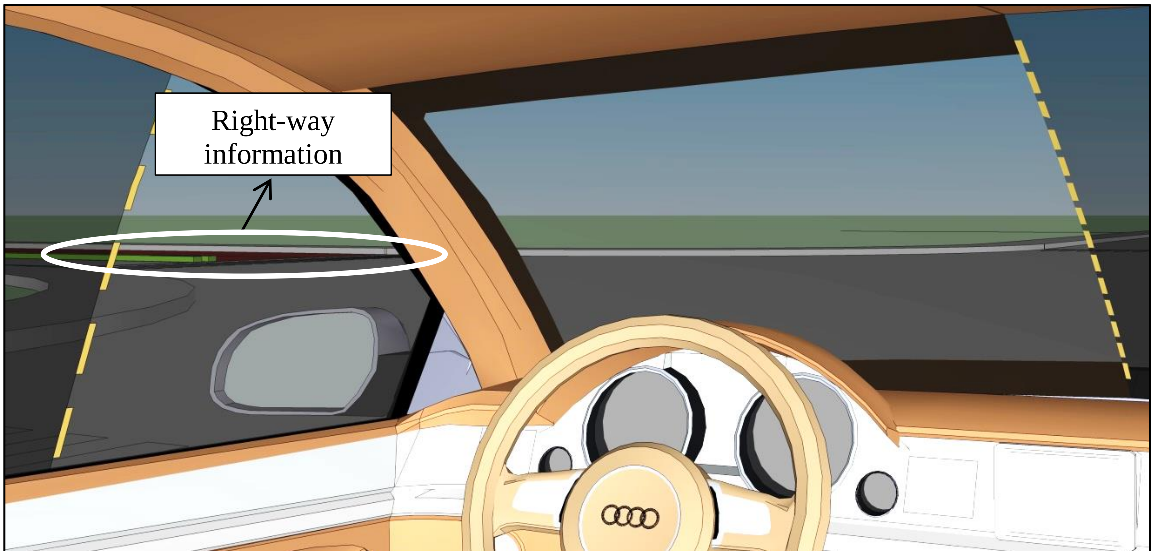


FIGURE 33 Example of driver's perspective analyses

Figure 34 illustrates an example of the comparison of information in a driver's perspective at the stop line positionings of 30%, 40%, 50%, and 60% with the same median barrier length and other geometric features based on the 3D analysis results. How right-way information and other roadway information change with different stop line positionings at the same parclo interchange terminal is observed in the new 3D analytical model of roadway environments.

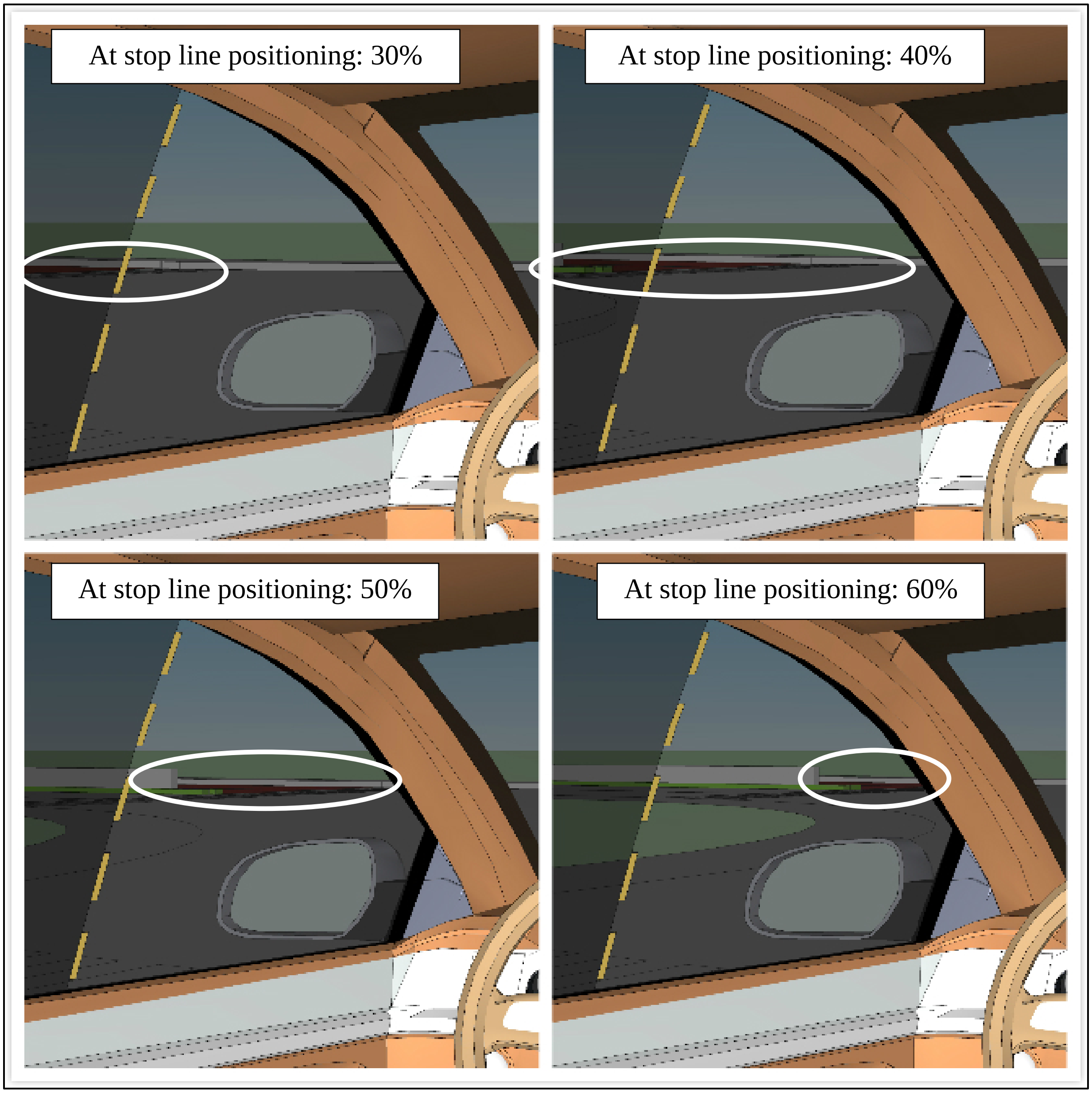


FIGURE 34 Comparison of the driver's perspectives at different stop line positionings

Parameter settings for the models were as follows: 73.66 cm (29 inches) was selected as the median barrier height in the model, as FHWA indicated the standard guardrail installation height was acceptable in the range from 70.48 cm (27.75 inches) to 76.20 cm (30 inches) above the ground based on crash testing and recommended 73.66 cm (29 inches) +/- 2.54 cm (1 inch) (*Manoj and Gautham, 2009*). The most common lane configuration was used: two lanes each for exit and entrance ramps and two through lanes with one one-way left-turn lane in each direction for the crossroad. Left-turn angles were 90 degrees. All roadways were level. Stop line positioning was selected as 30%, 40%, 50%, and 60% in each model. As median barriers should not extend on two-way ramps all the way to the stop line on ramps (*Morena and Leix, 2012*), the length of the median barrier was adjusted by gradually moving the ending point of the median barrier back from the terminal.

Figure 35 shows a comparison of information in a driver's perspective when moving the ending point of the median barrier back from the stop line on ramps by 1.5m, 3.0m, 4.5m, 6.0m, 7.5m, and 9.0m with the same intersection balance of 60% at the same parclo interchange terminal based on the 3D analysis results. From the figure, it can be seen that the right-way information in a driver's perspective increases as the length of the median barrier decreases by gradually moving the ending point of the median barrier back from the terminal.

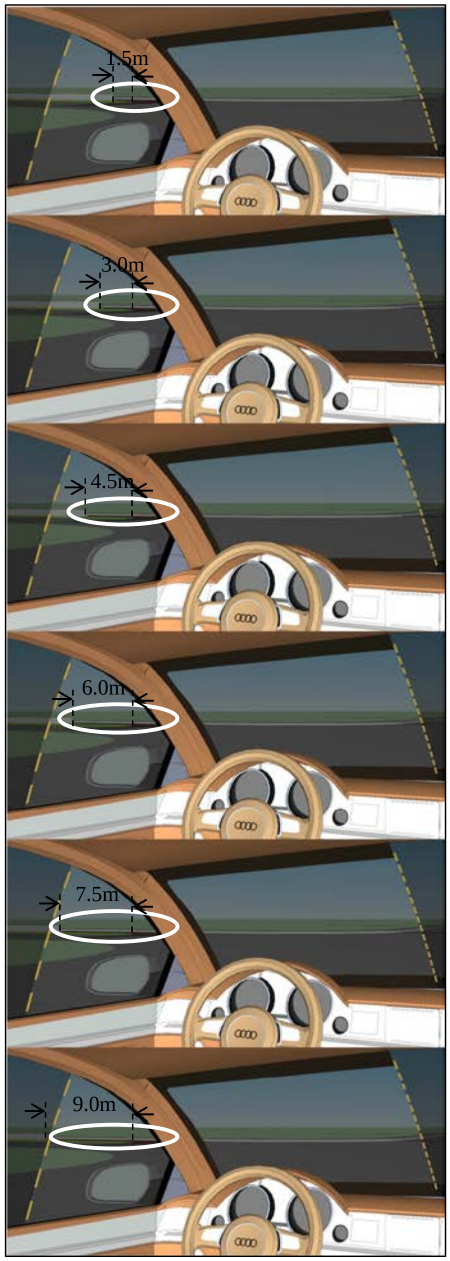


FIGURE 35 Comparison of the driver's perspectives when moving the ending point of the median barrier back from the stop line on ramps by 1.5m, 3.0m, 4.5m, 6.0m, 7.5m, and 9.0m from top to bottom

The initial ending point of the median barrier was set to be on the stop line of ramps. While moving the ending point back by 3.0m (9.84ft.) each time from 0.0m (0.00ft.) to 24.0m (78.74ft.), the percentage of right-way information was calculated (see Table 13). The right-way information in drivers' view increased with the reduction in median barrier length. Maximum right-way information was achieved at a certain length for each stop line positioning. For example, to maximize the right-way information in drivers' view when stop line positioning was 60%, the median barrier should be placed at least 21.0m (68.90ft.) away from the stop line on ramps; for stop line positioning of 50%, 40%, and 30%, the corresponding values were 18.0m (59.06ft.), 12.0m (39.37ft.), and 6.0m (19.69ft.), respectively.

TABLE 13. Right-way Information vs. Median Barrier Length and Stop Line Positioning

Right-way information percentage		**Distance from the median barrier ending point to the stop line on ramps**							
		3.0m (9.84ft.)	6.0m (19.69ft.)	9.0m (29.53ft.)	12.0m (39.37ft.)	15.0m (49.21ft.)	18.0m (59.06ft.)	21.0m (68.90ft.)	24.0m (78.74ft.)
Stop Line Positi-oning	30%	1.408%	**1.691%**	**1.691%**	**1.691%**	**1.691%**	**1.691%**	**1.691%**	**1.691%**
	40%	0.607%	0.895%	1.167%	**1.236%**	**1.236%**	**1.236%**	**1.236%**	**1.236%**
	50%	0.571%	0.621%	0.764%	0.876%	0.967%	**1.034%**	**1.034%**	**1.034%**
	60%	0.402%	0.498%	0.498%	0.564%	0.636%	0.700%	**0.754%**	**0.754%**

Note: Values in bold are the maximum that can be achieved.

6.4.4 Analysis of NDS data

Methodology

Vehicle speed, acceleration/deceleration rates, and videos of front and rear views were used to evaluate driver behavior. Statistical analyses were conducted to determine the mean, the variance, and the standard deviation as well as overall trend of vehicles' speed, acceleration/deceleration rates, and risk perception. Statistical hypothesis testing was performed to evaluate potential differences in average speed, deceleration rates, and braking distance between the interchange terminals with different intersection balances. Effectiveness of traffic control devices were quantified by video coding vehicles traveling from the cross road and then entering the entrance ramps.

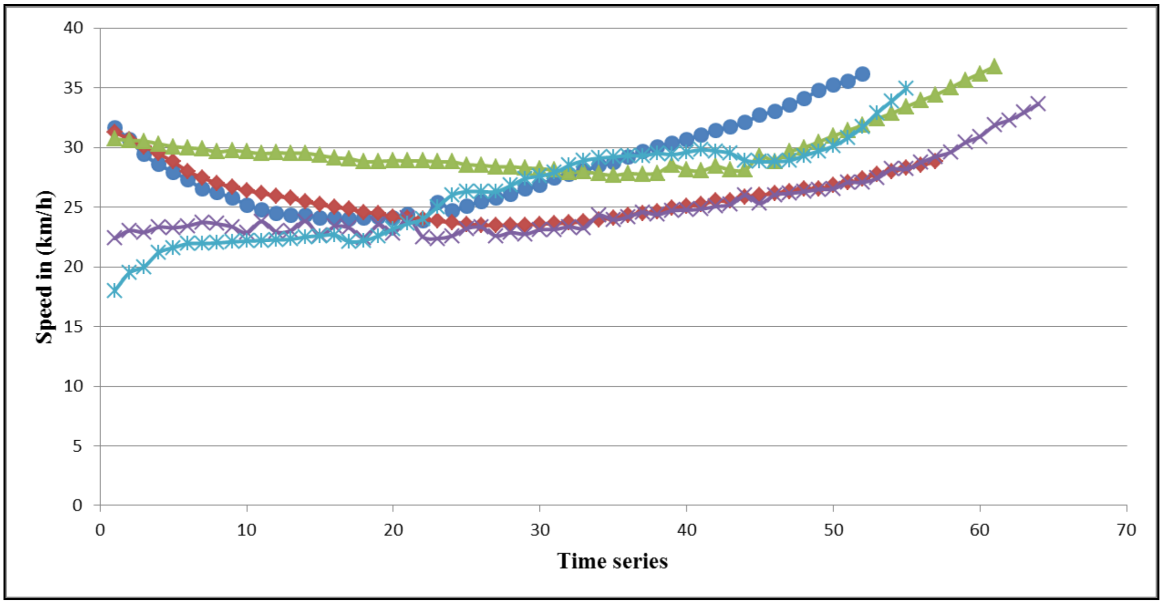
The collected NDS speed data are available at 0.1-s intervals from the NDS. Past analyses (*Oneyear et al., 2016; Machiani et al., 2016; Bao et al., 2015; Johnson et al., 2016; Guo et al., 2015; Oneyear et al., 2016; and Davis et al., 2015*) showed that one-second intervals were sufficient for speed data. Consequently, the data were aggregated and complete speed, acceleration/deceleration, and latitude/longitude information was determined at one-second intervals, which allow for an investigation of differences in driver speed selection by the intersection balance while controlling for other pertinent factors associated with each driving event. Thus, the collected data were utilized to investigate how driver speed and acceleration/deceleration selections vary among interchange terminals with different intersection balances; how intersection balances affect the standard deviation of speeds and acceleration/deceleration for drivers/vehicles during each event; and how driver risk perception, intersection balances, and speed and acceleration/deceleration related with each other.

To examine these questions, a cross-sectional analysis was used to compare compliant driver interactions from one study location with that of another study location to assess the safety effectiveness of the balanced intersection versus the unbalanced intersection: the higher the proportion of compliant behaviors observed, the better the safety performance of the intersection feature. Three primary metrics of interest were examined: 1) the average speed of vehicles during the time traveling at interchange terminals; 2) the variation in travel speeds for vehicles leading up to each event as quantified by the standard deviation of speeds over this period; and 3) the risk perception among study participants included in the events.

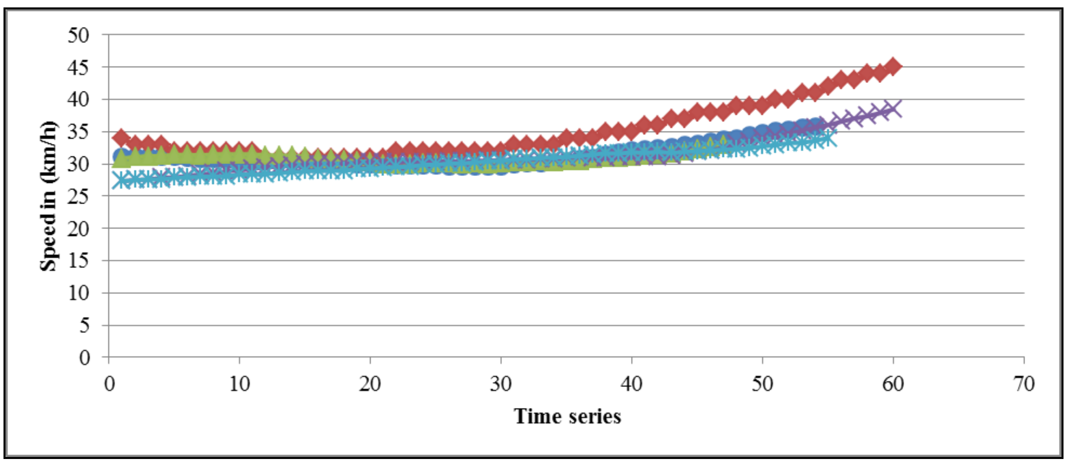
Results and Discussion

Speed and Acceleration/deceleration

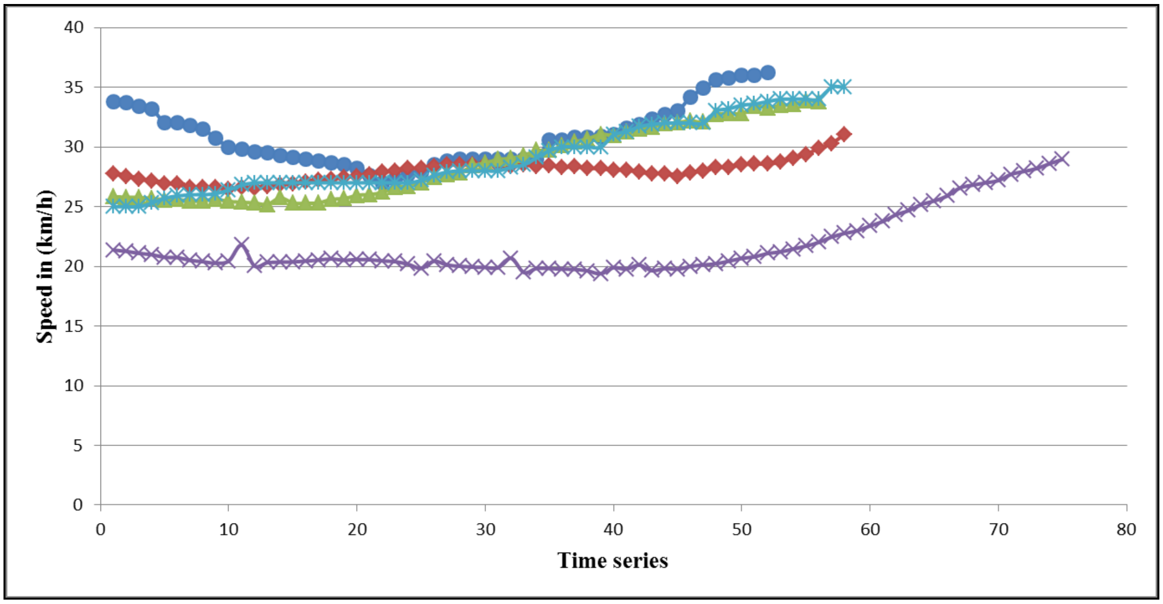
First, an explanatory analysis of the data was conducted to ascertain general trends in driver speed selection through the turning maneuver from the stop line on crossroads to entrance ramps. Speed and acceleration/deceleration were profiled for each selected trip at the three study sites. Before profiling, because all acceleration/deceleration data were in the unit of "g" in the NDS database, they were converted to the standard unit "m/s^2" for consistency with speed data. Sample speed and acceleration/deceleration profiles for five trips by five drivers are provided in Figure 36 and 37. Different lengths of the profiles were due to the different lengths of travel time as the speeds were different. The starting speeds of some trips were 0, because the vehicle stopped at red lights. Visual examination of these figures indicates significantly more variability in both the speed and the acceleration/deceleration profiles at unbalanced intersections, i.e. Location 1 (a signalized interchange terminal with the intersection balance of less than 50%) and Location 3 (a signalized interchange terminal with the intersection balance of more than 60%), compared with a balanced intersection, Location 2 (a signalized interchange terminal with the intersection balance in 50%-60%). Past research (*Fitzpathric et al., 2003; Garber and Ehrhart, 2000; Aljanahi et al., 1999; Garber and Gadiraju, 1988; and Arts and Van, 2006*) shows that drivers who maintain constant speeds tend to exhibit lower crash/near-crash risk. Combining the speed and acceleration/deceleration selections, which are the main indicators of drivers' comfort and confidence in driving, it shows that a balanced intersection of 50-60% accommodates traffic operation and safety.



(a) Sample speed profiles at Location 1

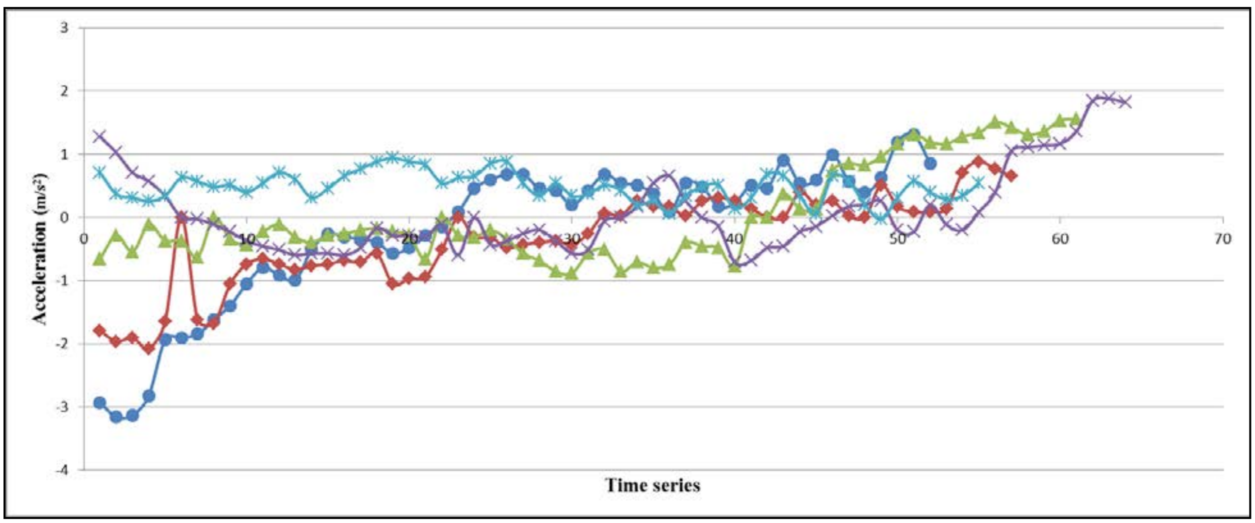


(b) Sample speed profiles at Location 2

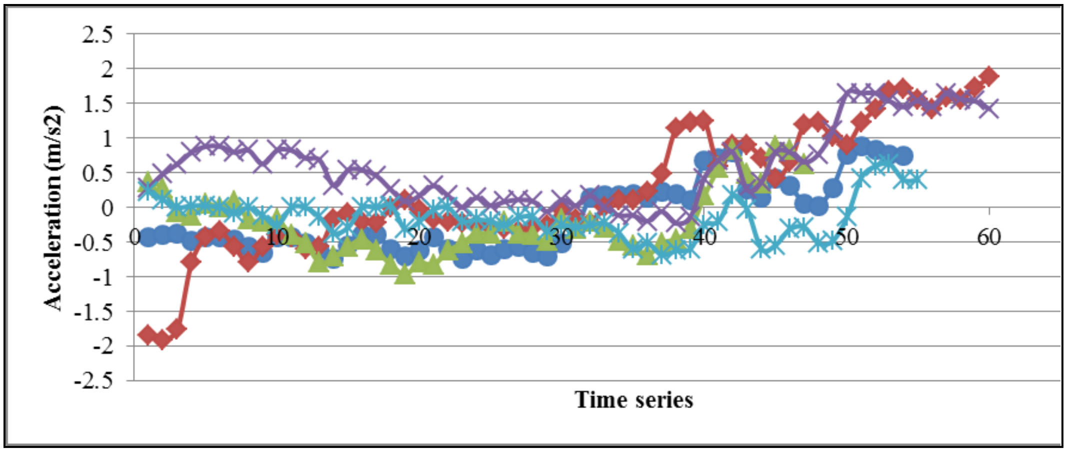


(c) Sample speed profiles at Location 3

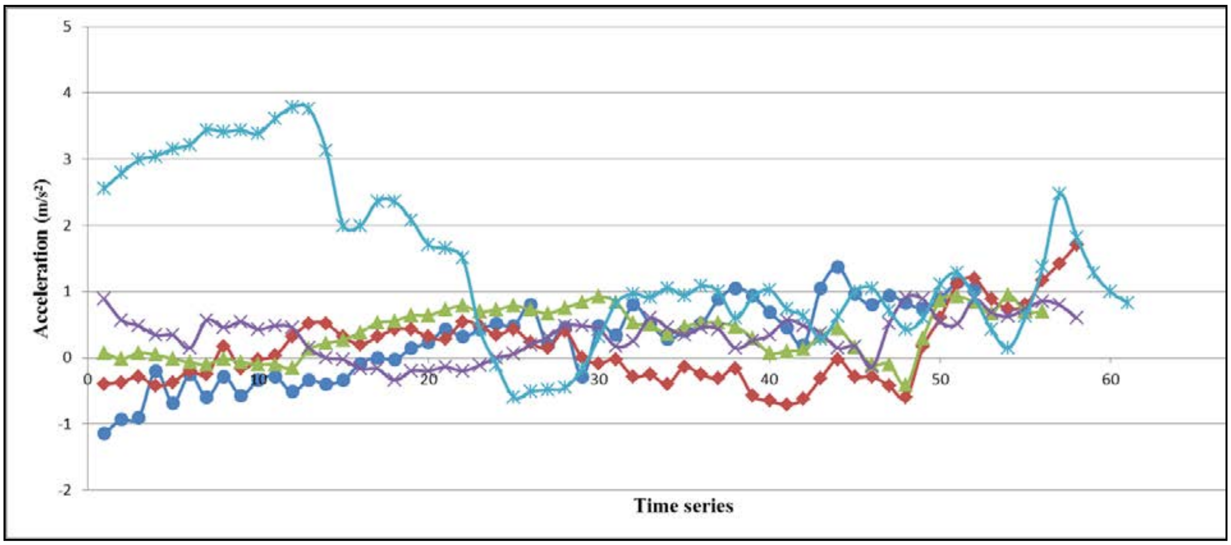
FIGURE 36 Sample speed profiles



(a) Sample s acceleration/deceleration profiles at Location 1



(b) Sample s acceleration/deceleration profiles at Location 2



(c) Sample s acceleration/deceleration profiles at Location 3

FIGURE 37 Sample acceleration/deceleration profiles

Table 14 provides summary statistics data related to the mean speeds among the 33 unique events in the final dataset using a 95% confidence interval as the relative measure at statistical significance level of 0.05. It provides details of the average values and the standard deviations of the mean speeds across the entire sample of events at each location. To clarify, the mean speed over the process transition from stop-line on crossroad to entrance ramps of each trip was calculated.

When examining these aggregate-level data, a few points stand out. First, the mean speeds tended to be relatively consistent when the intersection balance is larger than 50% and most concentrated when it is within 50%-60% for the standard deviation is the smallest. These mean speeds are also not found to be monotonically increasing, e.g., the mean speed at intersection balance of 51.43% is greater than at 73.33% of intersection balance. Secondly, it is also noted that the average and standard deviation of mean accelerations/decelerations of intersection balance of 50-60% are significantly smaller than the others, which indicates drivers were able to travel at a relatively very smooth transition through the intersection to entrance ramps, hence less crash risk. Thus, again it proves an intersection balance of 50-60% accommodate drivers mobility and safety the best.

TABLE 14 Summary Statistics of Mean Speed and Accelerations/Decelerations by Intersection Balance

Intersection Balance	Average of Mean Speeds	Std. Dev. of Mean Speeds	Average of Mean Accelerations/decelerations	Std. Dev. of Mean Accelerations/decelerations
38.78%	24.34	4.41	0.54	0.59
51.43%	32.28	2.60	0.08	0.24
73.33%	32.00	8.53	0.83	0.63

Table 15 provides similar summary statistics for the standard deviations in travel speeds over the duration of each left-turn trip. In contrast to the overall variability in speeds presented in Table 2, the standard deviations for individual trips are much smaller. This is reflective of the fact that driver speeds tend to be quite consistent. In general, speeds are shown to be more variable at unbalanced intersections when it is lower than 50% or larger than 60%, which is consistent with the speed profiles.

TABLE 15 Standard Deviations of Mean Speeds over Intersection Balance

Intersection Balance	Average of Std. Dev. Data
38.78%	5.96

51.43%	2.30
73.33%	9.45

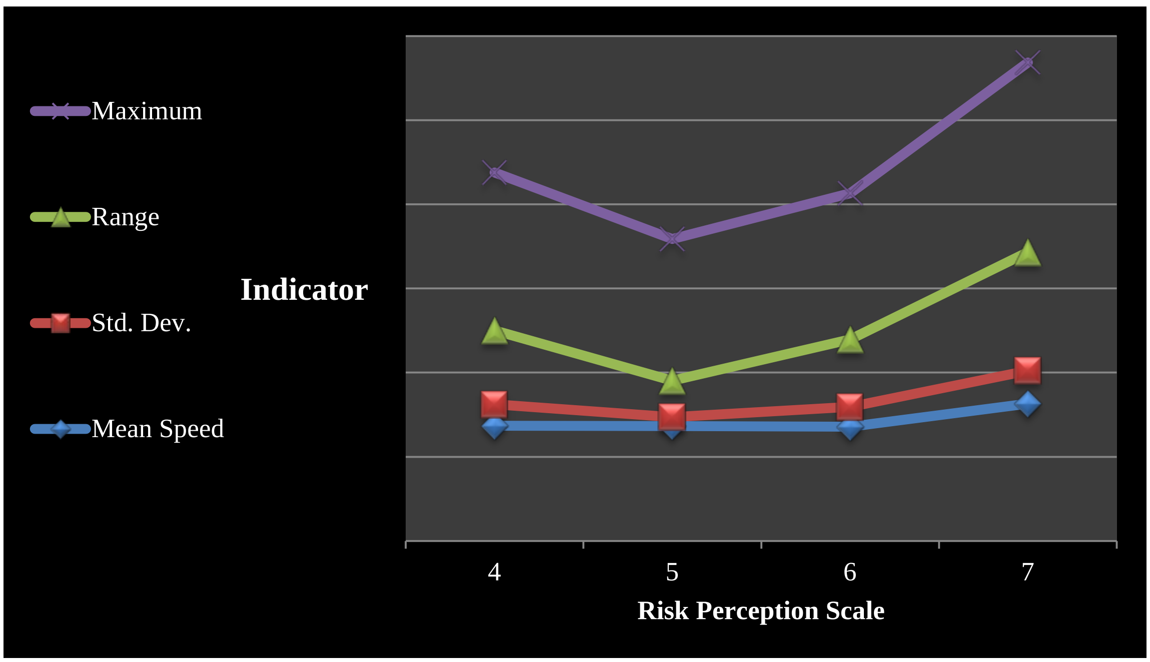
Risk Perception

To gain a better understanding of how driver risk perceptions affect driver speed and acceleration/deceleration selections, statistical analyses were conducted for speed and acceleration/deceleration as well as the standard deviations in them as described previously. The driver risk perceptions were obtained from the SHRP 2 NDS Risk Perception Questionnaire, a questionnaire designed to gauge the participant's perception of dangerous or unsafe driving behaviors or scenarios. The bigger the risk perception scale, the greater risk the driver is willing to take. For example, as defined in the data dictionary, a driver response of 4 to 7 indicates that a driver is willing to accept “Moderately Greater Risk” to “Much Greater Risk”. Afterwards, the relationship indicators of driver speeds and accelerations/decelerations were calculated using Equation 6 and 7, where a is the vehicle acceleration/deceleration in the longitudinal direction versus time., v is the vehicle velocity indicated on speedometer collected from network, m is the number of the drivers with the same risk perception, n is the number of trips with the same risk perception, and Y and Z are the relationship indicators of driver speeds and accelerations/decelerations respectively.

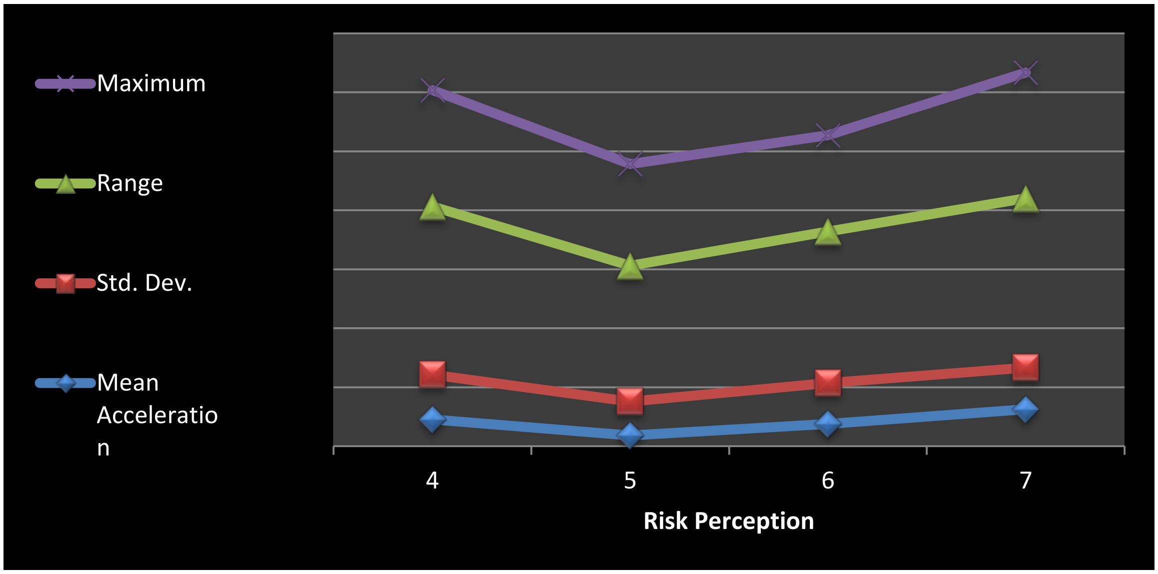
$$Y = \frac{1}{m}\sum_{i=0}^{m}\frac{1}{n}\sum_{\mathrm{i=0}}^{\mathrm{n}} v \qquad (6)$$

$$Z = \frac{1}{m}\sum_{i=0}^{m}\frac{1}{n}\sum_{\mathrm{i=0}}^{\mathrm{n}} a \qquad (7)$$

Figure 38 illustrates the relationships of speeds vs. risk perceptions (37a) and accelerations/decelerations vs. risk perceptions (37b). It demonstrates that there is no significant variance among driver speeds and accelerations/decelerations with increasing driver risk perception scale. The standard deviations in mean speeds and mean acceleration indicate as the standard deviations in mean speeds and accelerations/decelerations show that occurred during study events. Table 16 provides detailed results. The difference in speeds and acceleration among the four groups with four different risk perception scales were tested using a 95% confidence interval at the statistical significance of 0.05. It proves that no significant difference exists among the 4 groups. Thus, the risk perceptions of different drivers do not significantly influence the driver speeds and accelerations/decelerations in the collected dataset, i.e. there is no significant variance in speed and accelerations/decelerations due to driver risk perceptions.



(a) Speed vs. Risk Perception



(b) Acceleration/Deceleration vs. Risk Perception

FIGURE 38 Relationships of Speed vs. Risk Perception (a) and Acceleration/Deceleration vs. Risk Perception (b)

TABLE 16 Relationships of Speed vs. Risk Perception and Acceleration/Deceleration vs. Risk Perception

Relationship Indicator								
	Speed				Acceleration/Deceleration			
Risk Perception Scale	Mean Speed	Std. Dev.	Range	Maxi-mum	Mean Acceleration/deceleration	Std. Dev.	Range	Maxi-mum
4	27.41	5.06	17.55	37.56	0.45	0.76	2.85	1.97
5	27.29	2.16	8.67	33.62	0.18	0.58	2.30	1.76
6	27.18	4. 70	16.03	34.74	0.38	0. 70	2.57	1.63
7	32.59	7.88	28.19	45.07	0.63	0.70	2.87	2.13

6.5 Conclusion

At signalized intersections of two-way ramps and crossroads for a parclo interchange, the best current practice is to locate the stop line of left turns on cross roads at no more than 60% of the way through the intersection to provide an appropriate intersection balance for improving sight distance for traffic safety. Safety benefit of this practice has been proven based on WWD crash analysis in this study. The chance of WWD crashes increases significantly when the stop line is located more than 60% into the intersection.

Additionally, because the length of the median barrier on close-spaced two-way ramps has an important impact on a driver's perspective to determine the correct path, the effects of median barrier lengths on a driver's view was investigated using 3D simulation models at signalized intersections. With a decrease in median barrier length, the percentage of right-way information increases in the driver's sight and remains constant after the median barrier reaches a certain length. The median barrier should be extended for a certain distance from the stop line on ramps to expose the maximum entrance ramp information in the driver's sight. For example, drivers taking a left turn should have perceived the maximum entrance ramp information when the median barrier is located at 21m (70ft.), 18m (60ft.), 12m (40ft.), and 6m (20ft.) from the stop bar on the exit ramp for the intersection balance of 60%, 50%, 40%, and 30%, respectively. Guidelines can be developed for uniform median barrier lengths to improve sight distance at signalized ramp terminals of parclo interchanges. This study also showed that the 3D analytical methodology can be used to simulate drivers' view of roadway information. The results can be helpful in establishing design standards and guidelines with respect to the effects of other highway features on driver sight distance.

This paper also analyzed the statistical characteristics and overall trends of vehicles' speed, acceleration/deceleration rates, and risk perception. Intersection balance was observed to affect driver speed and acceleration, as well as a relationship between driver risk perception has and speed and acceleration/deceleration. A statistically significant difference was observed in mean speed, acceleration/deceleration rates, and the corresponding standard deviations at the study interchange terminals with different intersection balances. Drivers were found to adapt their speeds based upon changes in the roadway environment. Turning to the primary factor of interest, a balance intersection was found to result in smoother speeds and accelerations/decelerations, which are associated with lower crash/near-crash risks. Driver risk perceptions in the dataset were observed to not have a significant effect on the variability in driver travel speeds and accelerations/decelerations.

These findings are particularly noteworthy given the difficulty that is normally associated with relating driver perceptions with speed profile data. Most of the extant research literature has relied on traffic detector and crash data, which is often difficult to link directly to the time a maneuver happens due to time lags in these systems. These results provide compelling evidence that is based on real-time speed and acceleration/deceleration profiles.

Conclusively, at signalized interchange terminals of two-way ramps and crossroads, the stop line of left turns on cross roads should be located at no more than 60% of the way through the intersection to provide an appropriate intersection balance for better driver comfort and safety to improve traffic safety. The findings of this study are consistent with the extant literature (*Fitzpathric et al., 2003; Garber and Ehrhart, 2000; Aljanahi et al., 1999; Garber and Gadiraju, 1988; and Arts and Van, 2006*). Guidelines on design and application guidelines of intersection balance can be developed accordingly.

Future research can be conducted to evaluate the impacts of different approaching speeds of vehicles on drivers' sight; effects of median barrier lengths on drivers' sight at unsignalized intersections; effects of light at nighttime with headlight on drivers' view; how other geometric features in a roadway environment, such as number of lanes, different turning radii, and various slopes of roadways, influence the driver sight distance based on the methodology developed in this study; sight distance needs for older drivers (since older drivers may find it more difficult to turn their heads, necks, or upper bodies for an adequate line of sight down an acute-angle approach); analysis of high crash locations to explore which traffic control devices and roadway design features contribute more to prevent WWD crashes.

Additionally, the paper demonstrated a proof-of-concept for how naturalistic driving data could be leveraged to answer important safety questions of interest as to how drivers adapt their behavior in response to intersection balances. However, it reveals some limitations of the SHRP 2 NDS data: 1) The process for acquiring NDS data can take quite long; 2) turning movements are much less recorded than through movements in the NDS database; 3) The actual number of data applicable to a specific research question may be considerably smaller than the number of trips shown available on the NDS InSight Website.

CHAPTER 7 EFFECTS OF MEDIAN WIDTHS AT WRONG-WAY ENTRY POINTS

7.1 Introduction

In Alabama, wrong-way driving (WWD) crashes during 2009-2013 resulted in 18 fatalities in 14 fatal crashes (1.29 fatalities per fatal crashes), while this number for all divided highway fatal crashes was 1.13 (328 fatalities in 291 fatal crashes). More than 40% of WWD crashes led to severe injuries (Fatal and A-injury crashes), while only 7% of all crashes on divided highways in Alabama had severe injuries within the same time period, as vehicles' speeds on multilane divided highways are very high and WWD crashes are mostly head-on crashes, thus the probability of surviving is greatly declined. The percentage of WWD crashes during dark conditions (whether roadway is lit or not) is much higher than other divided highway crashes (71.1% vs. 21.3%). 45.5% of WWD crashes happened during 18:00-24:00, and 19.6% during (0:00-6:00). 60.7% of the vehicles involved were passenger cars.

The few other studies on WWD crashes on divided highways in other areas of the U.S. show similar trends. A study of wrong-way movements on divided, rural highways in Indiana (*1975*) revealed 39 deaths resulted from 96 WWD crashes over three years (1970-1972). Another WWD Crash analysis by the Arizona Department of Transportation (ADOT) (*2005*) indicated 91 people died and others were injured in 245 wrong-way crashes in Arizona over 10 years (2004-2014); 25% of wrong-way crashes were fatal compared to 1% of all crashes on divided highways.

Furthermore, Scifres et al.'s study (*1975*) indicated that about 40 percent of drivers making WWD entries emerged from intersections with crossroads on multilane divided highways. About 25 percent originated from business establishments such as gas stations and motels. About 20 percent originated from residential areas, crossovers, beginnings of divided sections, and construction sites, or were associated with U-turns and median openings. The origins of the remaining 15 percent were unknown.

The current breakpoint for median width, 30 feet, in the MUTCD (*2009*), is used to determine whether median openings are treated as one or two separate intersections for traffic control. The definition of intersection in Section 1A.13 of MUTCD (*2009*) indicates that crossings of two roadways 30 feet or more apart shall be considered two separate intersections. Figure 13 in a previous chapter shows the recommended wrong-way traffic control devices (TCDs) for divided highways with median widths of 30 feet or wider. The DO NOT ENTER (DNE) and WRONG WAY (WW) signs should be placed directly in view of road users at the point where road users could wrongly enter a divided highway as a supplement to physically discourage or prevent WWD.

Hence, WWD crashes lead to more severe crashes than non-WWD crashes multilane divided highways; most of them originated from median-crossroad intersections; darkness and crash time 18:00-6:00 were the leading crash scenario when sight distances could be compromised due to darkness and drivers impaired conditions. However, the impacts of varying median widths on wrong-way movements and sight distances at median openings have not been investigated. Besides, the foundation of current breakpoint for median widths for wrong-way TCD installations was based on transportation agency experience rather than scientific research. Therefore, this chapter intended to analyze the relationship between median widths and wrong-way incidents based on WWD crash and field data analyses for WWD prevention on multilane divided highways. Two types of data were collected: crash data on divided highways with varying median widths and field review data at study locations in Alabama. A two-step analysis approach was used: 1) analyzing the correlation between median widths and WWD crash occurrences; and 2) analyzing field data to determine effects of median widths on driver sight distance on crossroads, a critical factor for safety at the divided highway-crossroad intersections. Conclusions can guide traffic agencies for examining median-crossroad intersections and installing wrong-way TCDs based on median widths and grade changes to reduce wrong-way incidents on multilane divided highways.

7.2 Data Collection

7.2.1 Crash Data

The original crash database for a five-year period (2009-2013) was provided as hard-copy crash reports and electronic data by the Alabama Department of Transportation's Office of Safety Operations and the Critical Analysis Reporting Environment (CARE), including most of the variables in three separate files: crash, person, and vehicle. Altogether, 110 WWD crashes and 50,757 non-WWD crashes were collected on about 900-mile multilane divided highways in Alabama. The coordinates of the 110 WWD crash locations were extracted. Corresponding median widths were measured using existing online maps. The possible WWD entry point for each crash was estimated with the help of GIS capabilities of Google Map, and cross checked with the hard-copy police reports. Hence, 110 possible wrong-way entry median openings were identified

7.2.2 Field Data

Field experiments were conducted to determine driver sight distances from side streets at nighttime. The sight distance in this study was the distance that drivers can see with the vehicle's headlight illumination at median-crossroad intersections at nighttime. Figures 39 illustrates the drivers' wrong-way versus right-way movements. When a driver is completing a crossing maneuver, there must be sufficient sight distance in both directions available to cross the intersecting roadway and avoid approaching traffic. If the sight distance is shorter than the cross-section width of the median opening, the segment should be treated as two separate intersections (*FHWA, 2009*). The worst-case scenario for driver sight distance requirement was selected for the sight distance determination, which is the left-turn or through movement from the side streets. Given that most of WWD crashes occurred at nighttime, the sight distance was only checked for nighttime conditions. Sight distance adequacies were checked in the field with worst-case scenarios, i.e. drivers on crossroads in passenger cars with headlight illumination only at nighttime. By the AASHTO Green Book (*2011*), vehicles' average headlight span is 160-180 feet and drivers' average eye height in a passenger's car is 3.5 feet above the roadway. Both were verified for vehicles and drivers used in experiments. The experimental vehicle was passenger car, a 2010 Chrysler Sebring. The vehicle manual indicated that the headlights can reach approximately 160 feet during night time. Measurements were performed multiple times with a measurement wheel. The measured headlight span and driver's eye height were approximately 164 feet and 3.5 feet on average.

FIGURE 39 Drivers' wrong-way versus right-way movements on divided highways

Thirty-four sites in total with median widths distributed among 0 to 120 ft. were randomly selected from the 110 WWD entry points to conduct comprehensive field experiments.

Field trips were made on April 4, April 5, May 15, May 18, and May 19, 2015, during nighttime between 8:00 P.M. and 4:00 A.M. The first trip covered ten locations near Auburn AL, along a 63.9 mile segment from Milepost 120.249 to Milepost 135.605 on U.S. Highway 280. The second was to 5 selected locations in Montgomery AL, and the last included 19 locations in Birmingham and areas adjacent to Columbus, GA.

Detailed data of the roadway geometric features were collected on fields and cross checked using GIS capabilities of Google Earth Professional. For a study location, the measured parameters include: S = driver sight distance (ft.), G1, G2 = Grades in percent, G = Algebraic Grade Change in percent (%), M = Median width measured (MUTCD definition) (ft.), W = Width of lanes in one direction on the adjacent side of the stop bar (ft.), D = Distance from stop bar to the adjacent lane (ft.), O = Width of the opposite side roadways (ft.), L = Width of the entire cross-section of the median opening (ft.). The roadway elevations were determined based on the elevation profile of the vertical alignment of each median opening. The roadway grade was calculated using Equation (1).

Furthermore, the sight distance at drivers' eye height in a passenger car was evaluated in the field. Relevant data such as intersection controls, advanced signage, vertical curvature, and intersection conspicuity, which could not be clearly identified from online maps, were collected by field reviews. For each intersection approach, several more attributes were collected: 1) The distance for intersection recognizability: the distance to the intersection at which there are obvious indications (to multiple drivers) of the intersection presence, and if it is shorter than the driver sight distance; 2) Median type and width; and 3) Grades and vertical curvature (e.g. upgrade or downgrade, crest or sag vertical curve, etc.).

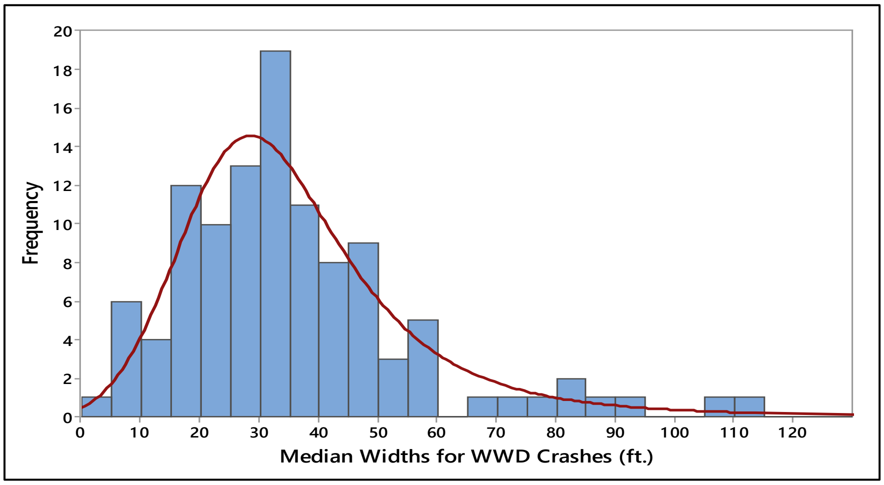
7.3 Data Analysis

7.3.1 Crash Data Analysis

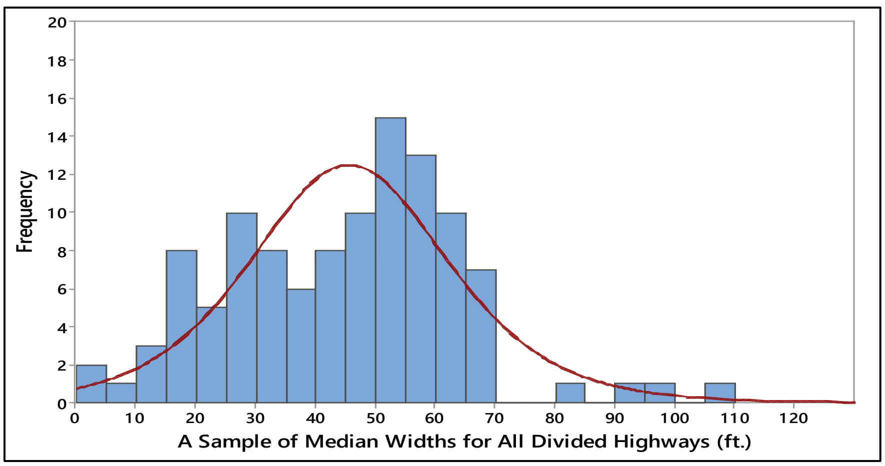
To evaluate the effects of median widths on WWD crashes, the distribution of median widths for 110 WWD crashes from 2009 to 2013 in Alabama were plotted in Figure 41a. The largest number of crashes occurred with median widths of 30-35 ft. The frequency fits a Log-logistic distribution as the red line showed at 95% confidence level.

By dividing 900 miles of divided highway into 0.1-mile increments, 9,000 data points were produced with different median widths, where a sample from median widths for all divided highways and another sample from median widths for 50,757 non-WWD crashes were randomly selected. These two frequency distributions are also best fit to a Log-logistic distribution (see figures 40b and 40c).

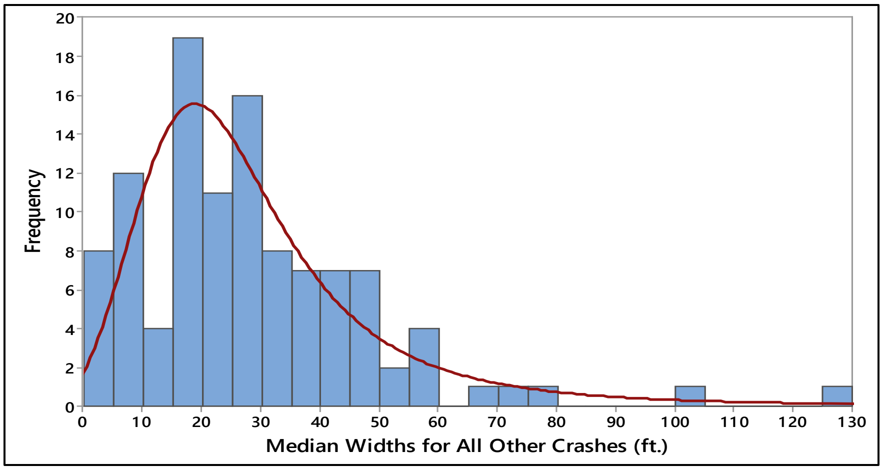
Two Mann-Whitney U tests were conducted to examine if there is a statistically significant difference between the distributions of median widths for WWD crashes and those for all divided highways/non-WWD crashes. This nonparametric test compares two unpaired samples to check if these two samples come from the same population. It works more efficiently than the t-test on continuous, non-normal distributions. The higher the value of mean rank, the higher the median width as the high score was associated with higher rank for median width when they were ordered for the test. For the two tests, the resultant Sum of Ranks of median widths for WWD crashes is 10,236.50, smaller than 14,073.50 for all divided highways and 13,753.50 for non-WWD crashes, while both corresponding p-values are less than 0.001. Both differences are statistically significant. The test results indicate that median width distributions are significantly different for WWD and non-WWD crashes on divided highways. However, crash data analysis cannot determine how median widths affect WWD crashes. The relationship between median widths effects and WWD crashes are further evaluated based on field data in the following section.



(a)



(b)



(c)

FIGURE 40 Frequency distributions of median widths for WWD crashes (a), of median widths for all divided highways (b), and of median widths for non-WWD crashes (c)

7.3.2 Field Data Analysis

Field experiments were conducted to determine driver sight distances from side streets at nighttime for the reevaluation of the current median width breakpoint. See Table 17 for the summary statistics of the collected field data for the selected 34 field review sites. It can be seen that all available sight distances are smaller than the headlight illumination distance of 164 feet. Thus, the sight distances were not constrained by the headlight illumination if there was no significant grade change.

TABLE 17 Summary Statistics of Field Data

Variable	**Observations**	**Minimum**	**Maximum**	**Mean**	**Std. deviation**
Sight Distance S (ft.)	34	34.76	158.58	105.37	30.29
Grade Change (%)	34	-27.27	18.54	-6.06	10.71
Median Width (ft.)	34	7.55	118.80	43.41	26.26
Total Lane Width (ft.)	34	21.72	58.66	28.97	9.82
Distance from stop bar to the adjacent lane (ft.)	34	8.66	43.46	20.13	9.49
Total cross-section width (ft.)	34	88.07	187.44	131.31	21.28

7.3.2.1 Correlation Analysis

A correlation analysis was conducted between the available sight distance (i.e. sight distance of a driver measured in the field) and the median features of study locations. This analysis investigated how median opening geometric features (i.e. the median width, grade change, distance from stop bar to the edge of adjacent travel lane, total lane width, and total cross-section width) affect the sight distance of drivers' from minor roads. The closer the result is to a value of one, the stronger the correlation. If it is negative, one factor increases as the other decreases (Table 18). It suggests that the driver's sight distance will be reduced with the increase in the median width and the grade change. It also shows that the median width has a significant negative correlation with the distance from stop bar to the edge of adjacent lane, which implies median widths decrease as distances from stop bar to the edge of adjacent lane increase. A significant positive correlation is seen between median width and total cross section widths.

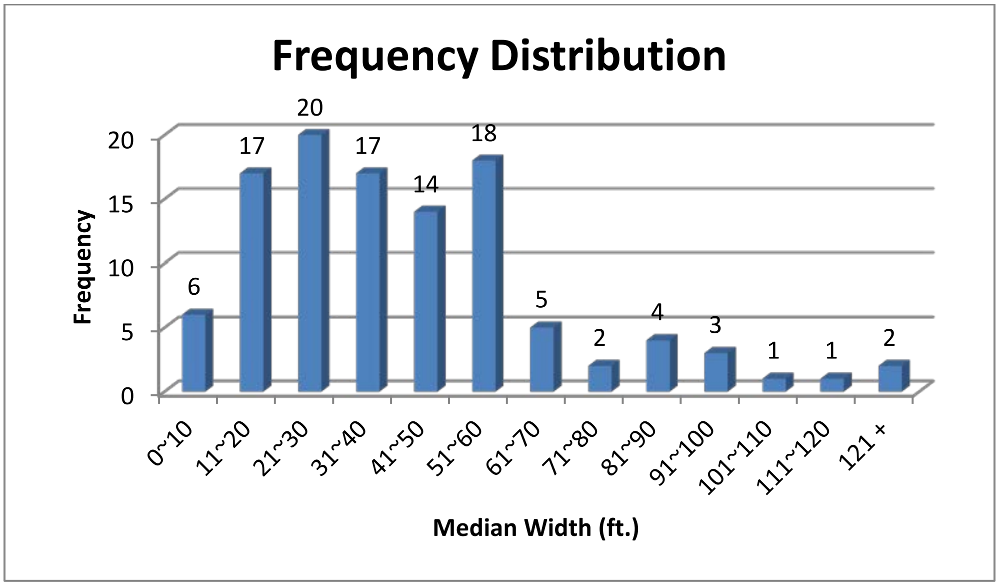
TABLE 18 Correlation Analysis Results

Variable	Available sight distance (ft.)	Grade change (%)	Distance from stop bar to the edge of adjacent lane (ft.)	Total lane width (ft.)	Median width (ft.)	Total cross-section width (ft.)
Available sight distance (ft.)	1					
Grade change (%)	**-0.204**	1				
Distance from stop bar to the edge of adjacent travel lane (ft.)	0.062	-0.256*	1			
Total lane width (ft.)	0.255*	-0.209*	0.178*	1		
Median width (ft.)	**-0.249**	0.129*	-0.526*	-0.495*	1	
Total cross-section width (ft.)	0.065	-0.108*	-0.020	0.044	0.630*	1

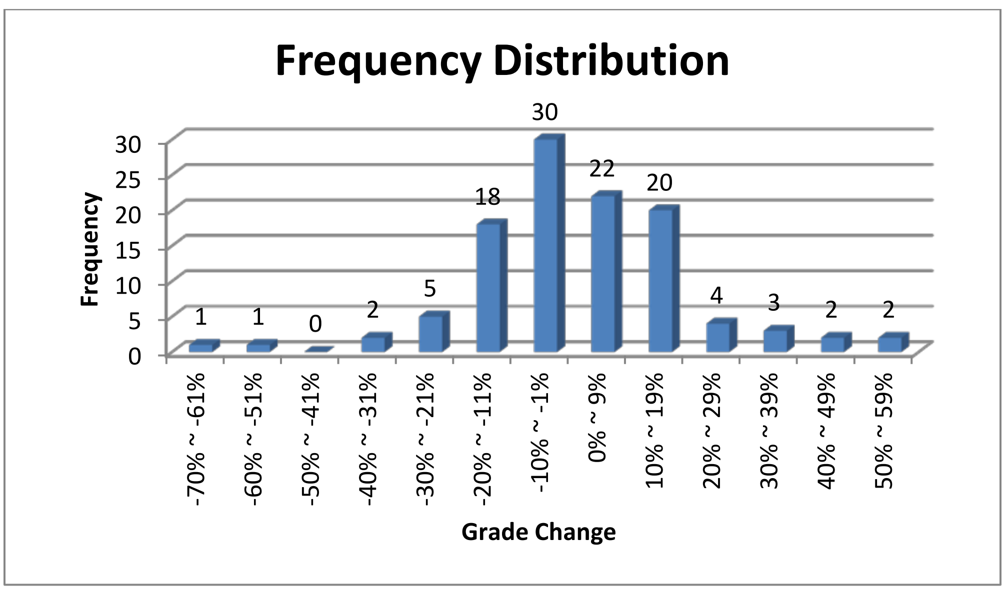
Note: *value is significant but not relevant to the interest of the study

Based on the correlation analysis results, two parameters, i.e. the median width and grade change, were chosen for further evaluation. The data on median widths and grade changes were collected for the 110 possible wrong-way entry median openings based on field review or

Google Earth. Figure 41a and 41b show the frequency distribution of the median widths and grade changes, respectively. It can be seen that most of the median widths range from 11 feet to 60 feet, and only a few are very narrow or wide medians; the values of grade changes concentrate on categories of -20% - -11%, -10% - -1%, 0% - 10%, and 11% - 20%.



(a)



(b)

FIGURE 41 Frequency distributions of median widths (a) and grade changes (b) for all 110 WWD entry points in Alabama

7.3.2.2 Sight Distance Determination

Furthermore, the Peto odds ratio (OR) assessed the effects of different median widths on sight distance using a 95% confidence interval as the relative measure at statistical significance level of 0.05. This calculation is based on the null hypothesis that the variable, median widths, has no effect on outcome, sight distance insufficiency. Under the null hypothesis, the difference between the observed and the expected would have zero difference and variance. The advantage of this Peto OR approach over direct calculations is that it allows for zero results without generating infinity, which is suitable for some categories of variables (i.e. median widths) in the dataset that have no events (i.e. occurrences of sight distance insufficiency). Thus, the odds of whether there would be sufficient sight distance at a certain range of median widths were calculated. The OR value reflects the impact of a specific category of median widths with larger numbers reflecting greater contribution. At statistical significance level of 0.05, significant differences exist between different categories of median widths. Results in Table 19 can reveal the different sufficiency of sight distances with different median widths. For example, compared with median widths of 10 ft. and less, the OR of 0.6315 indicates insufficient sight distance is less likely to occur at intersections with medians width within 21-30 ft.; when median widths are larger than 70ft., the likelihood of insufficient sight distance is 4 or more times higher. However, as all the confidence intervals contain 1.0, the analysis did not find those trends of sight distance sufficiency statistically significant, potentially due to the sample size.

TABLE 19 Peto OR Analysis Results for Effects of Median Widths on Sight Distances

Variable	**Category**	**OR**	**95% Confidence Interval**

Median Width	10 ft. and less	1 (Reference)	0.10 to 9.96
	11 to 20 ft.	1.63	0.20 to 12.93
	21 to 30 ft.	0.63	0.10 to 3.86
	31 to 40 ft.	1.63	0.20 to 12.92
	41 to 50 ft.	1.24	0.16 to 9.48
	51 to 60 ft.	1.29	0.18 to 9.47
	61 to 70 ft.	1.84	0.15 to 23.38
	71 to 80 ft.	4.74	0.15 to 150.31
	81 to 90 ft.	1.43	0.10 to 19.62
	91 to 100 ft.	5.55	0.24 to 128.67
	101 to 110 ft.	4.06	0.05 to 310.64
	111 to 120 ft.	4.06	0.05 to 310.64
	more than 121 ft.	4.74	0.15 to 150.31

Profiles of the entire cross section with driver sight in a passenger car were drawn for the 110 identified WWD entry points for sight distance determination. Figure 7 in chapter 3 illustrates an example. Besides the parameters defined early for field review, Ø, the Upswept angle of headlight beam from horizontal (typically 1 degree) was used (*AASHTO, 2011*). In each profile, the driver sight distance at the median opening was calculated based on the projection of driver sight on the profile of the entire cross section. The results were compared with the cross-section widths. If the sight distance is larger than the cross-section width, it is considered sufficient. Thus, the outcomes of the comparison are binary, denoted by 1 (the sight distance is sufficient) and 0 (otherwise).

7.4 Summary and Conclusions

This study evaluated the relationship between median widths and wrong-way crashes on multilane divided highways. Statistical analyses of crash data and field data were conducted to analyze the effects of median widths on and WWD entries and driver sight distance on multilane divided highways in Alabama. When determining intersection treatments, traffic agencies should depend on median widths, grade changes, as well as median opening profiling for sight distance sufficiency and WWD prevention, instead of just using median widths of 30 ft. as the breakpoint from MUTCD. Conclusions of the study can be used for traffic agencies to examine the sufficiency of wrong-way signing and driver sight distance at median-crossroad intersections on multilane divided highways. The dissertation validates the need and provides recommendations of MUTCD revisions.

Crash and field data in Alabama were collected on about 900-mile multilane divided highways. A five-year period (2009 -2013) crash data, i.e. 110 WWD crashes and 50,757 non-WWD crashes, were collected. Distribution analyses show median widths of 30-35 feet experienced most crashes. Mann-Whitney U tests indicated that median width distributions of WWD crashes are significantly different from other crashes.

Field experiments were conducted at 34 selected possible wrong-way entry median openings with median widths distributed among 7.55-118.80 ft. Correlation analysis suggested that the driver's sight distance was related primarily to the median width and grade change. So datasets of median widths, grade changes, and relevant vertical curvatures for all 110 possible wrong-way entry medians were further collected and processed for sight distance calculations and determination. The rationale behind this method is that if left-turn or through drivers from side streets cannot see the far side main roadways at nighttime, then it is necessary to treat the median opening as two intersections for wrong-way TCDs treatments. Peto odds ratio (OR) assessed the effects of different median widths on sight distance at statistical significance level of 0.05. Compared with median widths of 10 ft. and less, the OR of 0.6315 indicates insufficient sight distance is less likely to occur at intersections with medians width within 21-30 ft.; when median widths are larger than 70ft., the likelihood of insufficient sight distance is 4 or more times higher.

The chapter identified how median widths affect driver sight distance at nighttime with the influence of grade changes. When determining intersection treatments, both median widths and grade changes should be considered instead of just using 30 ft. as breakpoint from MUTCD. Results can help determine if the driver sight distance is sufficient at median-crossroad intersections based on different combinations of median widths and grade changes for installing TCDs to reduce wrong-way incidents on multilane divided highways.

CHAPTER 8 SUMMARY AND CONCLUSIONS

WWD is a major traffic safety hazard on roadways. It refers to driving movements against the direction of travel required by the traffic control devices (i.e., signs, pavement markings, and signals) along high-speed, physically-divided highways (i.e., principal arterials, expressways, freeways, and interstate highways) (*NHTSA, 2017*). WWD crashes are mostly head-on crashes at high speeds. Thus, the probability of surviving is much lower than other types of crashes.

To reduce WWD crashes, the drivers must be stopped from entering the wrong way in the first place. Improperly designed intersections can lead to operational and safety concerns along roadways when vehicles attempt to enter or exit roadways. WWD entries were mostly from median-crossroad intersections on roadways, where drive view of the intersection entrance can be restricted by various intersection features. Roadways with access that are not properly managed can experience lower safety. Therefore, the dissertation focused on intersection features at the wrong-way entry points, where drivers started to make a wrong-way maneuver and enter the wrong side of a roadway, addressing the effects of intersection features on WWD through evaluating the role of all components of the intersection traffic system that may restrict driver's view and maneuvers. Results indicate a need for a comprehensive systematic approach containing innovative safety countermeasures that are "practice ready" to reduce WWD entries.

The traditional "3Es of Traffic Safety", involve engineering, education, and enforcement. The engineering countermeasures typically focus on addressing roadway hazards; protecting drivers from the consequences of risky behaviors through designing better roads and vehicles. Similarly, education and enforcement countermeasures focus on the prevention of driver behavior, which may result in hazards, by training to encourage safe behaviors and punishing risky behaviors. Although the traditional 3E approach has been somewhat effective in reducing traffic fatalities, this approach alone cannot achieve a zero fatality vision. Hence, a traditional approach cannot eliminate WWD crashes on roadways; a multi-discipline (including a cross-discipline approach) is needed. It suggests the need for new safety countermeasures to reduce WWD crashes on roadways.

The research utilizes various types of data, including crash data, field data, survey data, simulation data, and NDS data, to investigate the effects of intersection features at wrong-way entry points on WWD. The study then evaluates how exactly each intersection feature affects driver view and maneuvers, including turn-prohibition signal control, signing and pavement marking traffic control devices, roadway geometric design elements, access management, intersection balance, length of median barrier, median type and width, as well as median opening treatments. Statistical analyses and 3D simulation analytics were used to accomplish the objectives of this research.

Chapter 3 documents an in-depth investigation of WWD crashes. This study developed and refined knowledge of the contributing factors and filled the gap of countermeasure of WWD, especially for Alabama. Extensive efforts were made on WWD crash analyses. The study developed various new engineering countermeasures that are recommended to be implemented in two phases. Phase one focuses on short-term, low-cost countermeasures, such as regular maintenance and inspection of signing, pavement markings, median modifications, and use of simple AM techniques. Phase two is a long-term, systematic approach for improving geometric design and ITS technologies, education, and enforcement. Transportation agencies should consider the causal factors for the WWD incidents in their jurisdictions and implement countermeasures that address the identified causes. Improvements with traffic signs and pavement markings are cheaper and easier to implement.

Chapter 4 evaluated the effectiveness and application circumstances of different combinations of the two signal indications for mitigating WWD**.** To deter WWD, Green Arrow (GA) or Circular Green (CG) indications are often used with No Right/Left Turn signs at intersections, but some researchers conjecture that the variety of CG and GA combinations might increase the likelihood of WWD. It is necessary to consider the safety implications of various signal displays and the difficulties in achieving uniformity across states. Factors to consider include driver understanding of the displays and safety effects. This study performed a comparison of GAs and CGs to deter WWD through surveys and interviews, particularly, including five different combinations of green signals and turn-prohibition signs. Recommendation on application and installation circumstances of signals for WWD prevention were made. In conclusion, replacing CGs with GAs can help prevent WWD. Turn-prohibition signs should complement GAs to enhance its performance and driver comprehension. LED turn-prohibition signs can be used with GAs at low-light, under-bridge, and high-crash-risk locations. This study fills the gap of identifying traffic signal countermeasures to reduce WWD movements at intersections.

Chapter 5 identifies possible geometric elements to direct driver movements at median-crossroad intersections and quantifies relationships between these geometric elements and WWD based on crash and field data analysis to help drivers' decision making and movements, which ultimately, improves traffic safety. The chapter involved an analysis of the collected WWD crash and geometric design data to quantify the relationships between geometric elements and WWD. This study establishes the foundation for countermeasure developments and design guide modifications to improve safety at wrong-way entry points. Accordingly, this study provides guidelines for intersection geometric design features as well as countermeasures for WWD prevention, including stop line positioning no more than 60%, turning Radii from crossroads to entrance roads less than 80 ft., using non-traversable median, and median widths less than 40 ft.

Chapter 6 concentrated on the effects of intersection balances on WWD, intersection sight distance, and driver behavior. As intersection balance is one essential design element in the planning, design, and operation of an intersection, a proper intersection balance can provide a safer and smother movement of left-turn vehicles while maintaining vehicular throughput and access to adjoining ramps. Despite the importance of intersection balance, it is often overlooked in current roadway design and site planning efforts. Thus, this study proves beneficial for improving freeway traffic safety and operational performance through controlling access location by proper intersection balance design using crash, simulation, and NDS and RID data. It also provides rationale for using the naturalistic data to quantify relationships between human and environmental factors for better roadway design, which are unknown from standard crash analysis. An optimal intersection balance design of 50% - 60 % is validated and recommended.

Chapter 7 investigated the impacts of varying median widths on wrong-way movements and sight distances at median openings. The foundation of current breakpoints for median widths for wrong-way TCD installations was based on transportation agency experience rather than scientific research. Therefore, the study intended to analyze the relationship between median widths and wrong-way incidents based on WWD crash and field data analyses for WWD prevention on physically divided highways. Two types of data were collected: crash data field review data at wrong-way entry points with varying median widths. A two-step analysis approach was used: 1) analyzing the correlation between median widths and WWD crash occurrences; and 2) analyzing field data to determine effects of median widths on driver sight distance on crossroads, a critical factor for safety at median-crossroad intersections. Conclusions can guide traffic agencies f to examine the sufficiency of wrong-way signing and driver sight distance at median-crossroad intersections. When determining intersection treatments, traffic agencies should depend on median widths, grade changes, as well as median opening profiling for sight distance sufficiency and WWD prevention, instead of just using median widths of 30 ft. as the breakpoint from MUTCD.

This research developed innovative countermeasures to deter drivers from entering wrong way at intersections where WWD originated. It demonstrates “practice ready” countermeasures to address intersection sight distance issues to help drivers with decision making and maneuvers to prevent drivers from entering wrong ways in the first place. Corresponding recommendations are made on best design practice and guidelines that may discourage wrong-way entries, which, ultimately, reduce WWD crashes.

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APPENDIX A: CONTRIBUTING FACTOR CUMULATIVE FREQUENCY

TABLE A.1 Pre-Crash/Human – Fatal Crashes

Pre-Crash: Human			
Age of Driver	16-24	2	15.38%
	25-34	0	0.00%
	35-44	3	23.08%
	45-54	0	0.00%
	55-64	3	23.08%
	Above 65	5	38.46%
Gender of Driver	Male	9	69.23%
	Female	4	30.77%
Driver Condition	Under the Influence of Alcohol/Drugs	1	7.69%
	Other/Unknown	12	92.31%
Driver Contributing Circumstance	Traveling Wrong Way/Wrong Side	11	84.62%
	Other/Unknown	1	7.69%
	DUI	1	7.69%

TABLE A .2 Pre-Crash/Human – A-Injury Crashes

Pre-Crash: Human			
Age of Driver	16-24	2	9.09%
	25-34	5	22.73%
	35-44	5	22.73%
	45-54	2	9.09%
	55-64	1	4.55%
	Above 65	7	31.82%
Gender of Driver	Male	13	59.09%
	Female	8	36.36%
	NA	1	4.55%
Driver Condition	Under the Influence of Alcohol/Drugs	9	40.91%
	Apparently Normal	8	36.36%
	Asleep, fainted, fatigued, etc.	0	0.00%
	Emotional (Depressed/Angry/Disturbed)	1	4.55%
	Illness	1	4.55%
	Other/Unknown	3	13.64%
Driver Contributing Circumstance	Traveling Wrong Way/Wrong Side	18	81.82%
	Other/Unknown	2	9.09%
	Driver Condition	1	4.55%
	DUI	1	4.55%

TABLE A.3 Pre-Crash/Human – B-Injury Crashes

Pre-Crash: Human			
Age of Driver	16-24	2	11.76%
	25-34	3	17.65%
	35-44	2	11.76%
	45-54	3	17.65%
	55-64	2	11.76%
	Above 65	4	23.53%
	Unknown	1	5.88%
Gender of Driver	Male	10	58.82%
	Female	6	35.29%
	NA	1	5.88%
Driver Condition	Under the Influence of Alcohol/Drugs	7	41.18%
	Apparently Normal	7	41.18%
	Other/Unknown	3	17.65%
Driver Contributing Circumstance	Traveling Wrong Way/Wrong Side	12	70.59%
	Driver Condition	1	5.88%
	DUI	1	5.88%
	NA or Other	3	17.65%

TABLE A.4 Pre-Crash/Vehicle – Fatal Crashes

Pre-Crash: Vehicle			
Vehicle 1 Maneuver	Movement Essentially Straight	12	92.31%
	Wrong Way on One Way	1	7.69%
Vehicle 1 Type	Passenger Car	8	61.54%
	Pick-up	1	7.69%
	SUV	2	15.38%
	Van	2	15.38%
Vehicle 2 Maneuver	Movement Essentially Straight	13	100.00%
Vehicle 2 Type	Passenger Car	8	61.54%
	SUV	1	7.69%
	Pick-up	2	15.38%
	Single Unit Truck/Other Light Truck	2	15.38%

TABLE A.5 Pre-Crash/Vehicle – A-Injury Crashes

Pre-Crash: Vehicle			
Vehicle 1 Maneuver	Movement Essentially Straight	18	81.82%
	Wrong Side of Road	3	13.64%
	Making U-Turn	1	4.55%
Vehicle 1 Type	Passenger Car	11	50.00%
	Pick-up	10	45.45%
	SUV	1	4.55%
Vehicle 2 Maneuver	Movement Essentially Straight	18	81.82%
	Stopped for Sign/Signal	1	4.55%
	Slowing/Stopping	1	4.55%
	No Second Vehicle	1	4.55%
	Other	1	4.55%
Vehicle 2 Type	Passenger Car	8	36.36%
	Pick-up	8	36.36%
	SUV	4	18.18%
	Tractor/semi-trailer	1	4.55%
	Single-Unit Truck	1	4.55%

TABLE A.6 Pre-Crash/Vehicle – B-Injury Crashes

Pre-Crash: Vehicle			
Vehicle 1 Maneuver	Movement Essentially Straight	15	88.24%
	Turning Left	1	5.88%
	Other	1	5.88%
Vehicle 1 Type	Passenger Car	12	70.59%
	Pick-up	2	11.76%
	SUV	3	17.65%
Vehicle 2 Maneuver	Movement Essentially Straight	13	76.47%
	Stopped in Traffic	1	5.88%
	Turning Left	2	11.76%
	Other	1	5.88%
Vehicle 2 Type	Passenger Car	12	70.59%
	Pick-up	2	11.76%
	Tractor/semi-trailer	1	5.88%
	SUV	1	5.88%
	Motorcycle	1	5.88%

TABLE A.7 Crash/Human – Fatal Crashes

Crash: Human			
Contributing Circumstance	Traveling Wrong Way/Wrong Side	13	100.00%

TABLE A.8 Crash/Human – A-Injury Crashes

Crash: Human			
Contributing Circumstance	Traveling Wrong Way/Wrong Side	13	59.09%
	DUI	9	40.91%

TABLE A.9 Crash/Human – B-Injury Crashes

Crash: Human			
Contributing Circumstance	Traveling Wrong Way/Wrong Side	12	70.59%
	Improper Lane Change/Use	1	5.88%
	DUI	4	23.53%

TABLE A.10 Crash/Vehicle – Fatal Crashes

Crash: Vehicle			
Type of Crash	Head-on	9	69.23%
	Angle Opposite Direction	1	7.69%
	Angle Oncoming (Frontal)	1	7.69%
	Single Vehicle Crash	1	7.69%
	Sideswipe - Opposite Direction	1	7.69%
Air Bag	Deployed Front	9	69.23%
	Deployed Multiple Combinations	2	15.38%
	Deployed	1	7.69%
	Not Installed	1	7.69%
Safety Equipment	Seat Belts Used	8	61.54%
	Seat Belts Not Used	5	38.46%

TABLE A.11 Crash/Vehicle – A-Injury Crashes

Crash: Vehicle			
Type of Crash	Head-on	11	50.00%
	Non-collision	1	4.55%
	Side Impact (Angled)	2	9.09%
	Angle Oncoming (Frontal)	4	18.18%
	Sideswipe, Opposite Direction	2	9.09%
	Angle Opposite Direction	2	9.09%
Air Bag	Deployed Front	14	63.64%
	Not Deployed	3	13.64%
	Deployed	2	9.09%
	Unknown	3	13.64%
Safety Equipment	Seat Belts Used	14	63.64%
	Seat Belts Not Used	5	22.73%
	Unknown	3	13.64%

TABLE A.12 Crash/Vehicle – B-Injury Crashes

Crash: Vehicle			
Type of Crash	Head-on	11	64.71%
	Angle Opposite Direction	2	11.76%
	Side Impact (Angled)	1	5.88%
	Single Vehicle Crash	1	5.88%
	Sideswipe, opposite direction	1	5.88%
	Other	1	5.88%
Air Bag	Deployed Front	9	52.94%
	Not Deployed	4	23.53%
	Deployed	1	5.88%
	Not Installed	1	5.88%
	Unknown	2	11.76%
Safety Equipment	Seat Belts Used	14	82.35%
	Seat Belts Not Used	2	11.76%
	NA	1	5.88%

TABLE A.13 Crash/Environment – Fatal Crashes

Crash: Environment			
Roadway Surface	Dry	12	92.31%
	Wet	1	7.69%
Light Condition	Darkness-Road Not Lit	9	69.23%
	Darkness-Road Lit	2	15.38%
	Daylight	2	15.38%
Weather	Clear	10	76.92%
	Cloudy	3	23.08%
Work Zone	Not In/Related	12	92.31%
	Yes	1	7.69%

TABLE A.14 Crash/Environment – A-Injury Crashes

Crash: Environment			
Roadway Surface	Dry	19	86.36%
	Wet	3	13.64%
Light Condition	Darkness-Road Not Lit	12	54.55%
	Darkness-Road Lit	4	18.18%
	Daylight	6	27.27%
Weather	Clear	18	81.82%
	Cloudy	1	4.55%
	Rain	3	13.64%
Work Zone	Not In/Related	19	86.36%
	NA	1	4.55%
	Between Warning Signs and Work Area	2	9.09%

TABLE A.15 Crash/Environment – B-Injury Crashes

Crash: Environment			
Roadway Surface	Dry	12	70.59%
	Wet	5	29.41%
Light Condition	Darkness-Road Not Lit	12	70.59%
	Darkness-Road Lit	2	11.76%
	Daylight	3	17.65%
Weather	Clear	8	47.06%
	Cloudy	6	35.29%
	Rain	3	17.65%
Work Zone	Not In/Related	16	94.12%
	NA	1	5.88%

TABLE A.16 Pre-Crash/Human – Fatal and A-Injury Crashes

Pre-Crash: Human			
Age of Driver	16-24	4	11.43%
	25-34	5	14.29%
	35-44	8	22.86%
	45-54	2	5.71%
	55-64	4	11.43%
	Above 65	12	34.29%
Gender of Driver	Male	22	62.86%
	Female	12	34.29%
	NA	1	2.86%
Driver Condition	Under the Influence of Alcohol/Drugs	10	28.57%
	Apparently Normal	8	22.86%
	Asleep, fainted, fatigued, etc.	0	0.00%
	Emotional (Depressed/Angry/Disturbed)	1	2.86%
	Illness	1	2.86%
	Other/Unknown	15	42.86%
Driver Contributing Circumstance	Traveling Wrong Way/Wrong Side	29	82.86%
	Other/Unknown	3	8.57%
	Driver Condition	1	2.86%
	DUI	2	5.71%

TABLE A.17 Pre-Crash/Vehicle – Fatal and A-Injury Crashes

Pre-Crash: Vehicle			
Vehicle 1 Maneuver	Movement Essentially Straight	30	85.71%
	Wrong Side of Road	3	8.57%
	Wrong Way on One Way	1	2.86%
	Making U-Turn	1	2.86%
Vehicle 1 Type	Passenger Car	19	54.29%
	Pick-up	11	31.43%
	SUV	3	8.57%
	Van	2	5.71%
Vehicle 2 Maneuver	Movement Essentially Straight	31	88.57%
	Stopped for Sign/Signal	1	2.86%
	Slowing/Stopping	1	2.86%
	Other	1	2.86%
	No Second Vehicle	1	2.86%
Vehicle 2 Type	Passenger Car	16	45.71%
	Pick-up	10	28.57%
	SUV	5	14.29%
	Tractor/semi-trailer	1	2.86%
	Single-Unit Truck	3	8.57%

TABLE A.18 Crash/Human – Fatal and A-Injury Crashes

Crash: Human			
Contributing Circumstance	Traveling Wrong Way/Wrong Side	26	74.29%
	DUI	9	25.71%

TABLE A.19 Crash/Vehicle – Fatal and A-Injury Crashes

Crash: Vehicle			
Type of Crash	Head-on	20	57.14%
	Angle Opposite Direction	3	8.57%
	Angle Oncoming (Frontal)	5	14.29%
	Single Vehicle Crash	1	2.86%
	Sideswipe - Opposite Direction	3	8.57%
	Side Impact (Angled)	2	5.71%
	Non-collision	1	2.86%
Air Bag	Deployed Front	23	65.71%
	Not Deployed	3	8.57%
	Deployed Multiple Combinations	2	5.71%
	Deployed	3	8.57%
	Not Installed	1	2.86%
	Unknown	3	8.57%
Safety Equipment	Seat Belts Used	22	62.86%
	Seat Belts Not Used	10	28.57%
	NA	3	8.57%

TABLE A.20 Crash/Environment – Fatal and A-Injury Crashes

Crash: Environment			
Roadway Surface	Dry	31	88.57%
	Wet	4	11.43%
Light Condition	Darkness-Road Not Lit	21	60.00%
	Darkness-Road Lit	6	17.14%
	Daylight	8	22.86%
Weather	Clear	28	80.00%
	Cloudy	4	11.43%
	Rain	3	8.57%
Construction Zone	Not In/Related	31	88.57%
	Yes	1	2.86%
	NA	1	2.86%
	Between Warning Signs and Work Area	2	5.71%

TABLE A.21 Pre-Crash/Human – Fatal, A-, and B-Injury Crashes

Pre-Crash: Human			
Age of Driver	16-24	6	11.54%
	25-34	8	15.38%
	35-44	10	19.23%
	45-54	5	9.62%
	55-64	6	11.54%
	Above 65	16	30.77%
	Unknown	1	1.92%
Gender of Driver	Male	32	61.54%
	Female	18	34.62%
	NA	2	3.85%
Driver Condition	Under the Influence of Alcohol/Drugs	17	32.69%
	Apparently Normal	15	28.85%
	Emotional (Depressed/Angry/Disturbed)	1	1.92%
	Illness	1	1.92%
	Other/Unknown	18	34.62%
Driver Contributing Circumstance	Traveling Wrong Way/Wrong Side	41	78.85%
	Driver Condition	2	3.85%
	DUI	3	5.77%
	Other/Unknown	6	11.54%

TABLE A.22 Pre-Crash/Vehicle – Fatal, A-, and B-Injury Crashes

Pre-Crash: Vehicle			
Vehicle 1 Maneuver	Movement Essentially Straight	45	86.54%
	Wrong Side of Road	3	5.77%
	Wrong Way on One Way	1	1.92%
	Making U-Turn	1	1.92%
	Turning Left	1	1.92%
	Other	1	1.92%
Vehicle 1 Type	Passenger Car	31	59.62%
	Pick-up	13	25.00%
	SUV	6	11.54%
	Van	2	3.85%
Vehicle 2 Maneuver	Movement Essentially Straight	44	84.62%
	Stopped for Sign/Signal	2	3.85%
	Slowing/Stopping	1	1.92%
	Turning Left	2	3.85%
	Other	2	3.85%
	No Second Vehicle	1	1.92%
Vehicle 2 Type	Passenger Car	28	53.85%
	Pick-up	12	23.08%
	SUV	6	11.54%
	Tractor/semi-trailer	2	3.85%
	Motorcycle	1	1.92%
	Single-Unit Truck	3	5.77%

TABLE A.23 Crash/Human – Fatal, A-, and B-Injury Crashes

Crash: Human			
Contributing Circumstance	Traveling Wrong Way/Wrong Side	38	73.08%
	Improper Lane Change/Use	1	1.92%
	DUI	13	25.00%

TABLE A.24 Crash/Vehicle – Fatal, A-, and B-Injury Crashes

Crash: Vehicle			
Type of Crash	Head-on	31	59.62%
	Angle Opposite Direction	5	9.62%
	Angle Oncoming (Frontal)	5	9.62%
	Single Vehicle Crash	2	3.85%
	Sideswipe - Opposite Direction	4	7.69%
	Side Impact (Angled)	3	5.77%
	Other	1	1.92%
	Non-collision	1	1.92%
Air Bag	Deployed Front	32	61.54%
	Not Deployed	7	13.46%
	Deployed Multiple Combinations	2	3.85%
	Deployed	4	7.69%
	Not Installed	2	3.85%
	Unknown	5	9.62%
Safety Equipment	Seat Belts Used	36	69.23%
	Seat Belts Not Used	12	23.08%
	NA	4	7.69%

TABLE A.25 Crash/Environment – Fatal, A-, and B-Injury Crashes

Crash: Environment			
Roadway Surface	Dry	43	82.69%
	Wet	9	17.31%
Light Condition	Darkness-Road Not Lit	33	63.46%
	Darkness-Road Lit	8	15.38%
	Daylight	11	21.15%
Weather	Clear	36	69.23%
	Cloudy	10	19.23%
	Rain	6	11.54%
Work Zone	Not In/Related	47	90.38%
	Yes	1	1.92%
	NA	2	3.85%
	Between Warning Signs and Work Area	2	3.85%