Safety Comparisons Between Interchange Types

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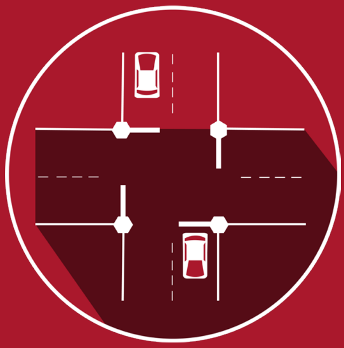

U.S. Department of Transportation
Federal Highway Administration

Research, Development, and Technology
Turner-Fairbank Highway Research Center
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McLean, VA 22101-2296

FOREWORD

The *Highway Safety Manual* has made predictive safety analysis feasible for many basic roadway elements, including freeway and ramp segments and ramp terminals.(3) This capability raises the expectation for predicting the safety performance of more complex roadway facilities, such as interchanges. Although interchanges can be decomposed into basic road elements, the safety performance of interchanges cannot be derived by simply adding predictions from individual components.

The Federal Highway Administration (FHWA) supported a project to explore planning-level analysis of interchange configurations during alternatives analysis or Interchange Access Requests (IARs). FHWA sought to identify the most commonly considered configurations in IARs and develop a predictive model and implementation tool. The purpose of this predictive model and associated implementation tool is to use an approach with more robust considerations than a single crash modification factor and provide reliable predictions using information commonly known during interchange project planning. The model and implementation tool can be used to evaluate the predicted crash frequency and severity for interchange configurations under consideration using basic inputs for the entire interchange area.

This report documents the methods and processes used to identify applicable service interchange configurations for inclusion in the predictive method and provides details on data collection procedures and methods used for developing crash frequency and severity implementation models. The results of this research can be used to support planning-level analysis of various interchange configurations under consideration early in the project development process.

Brian, Cronin, P.E.
Director, Office of Safety and Operations
Research and Development

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Documentation of potential safety performance impacts is required for Interchange Justification Reports (IJRs) to validate the need for new or modified interchanges.[2] Typically, IJRs are written early in the project planning and design process, with details generally consistent with conceptual design. As such, the design details required for using the Highway Safety Manual (HSM) Part C predictive models may not be known, only the general form of the interchanges.[1] Without these details, using the HSM models to develop an accurate prediction of crash frequency and severity of the design is difficult. In addition, aggregating site-by-site predictions may not fully capture the safety performance impacts when considering the project location as a whole. The Federal Highway Administration sought to develop planning-level models and tools to predict crash frequency and severity for an existing or proposed interchange to further explore and address an IJR application. This predictive model estimates crash frequency and severity for service interchange configurations comprising more than 75 percent of those typically considered in IJRs. The safety performance function includes freeway and crossroad annual average daily traffic (AADT) per lane, ramp AADT, and interchange configuration. The base estimate is adjusted for freeway lanes, crossroad lanes, area type, interchange skew, a nearby adjacent interchange gore, managed lanes, the number of crossroad left-turn lanes at terminals, and variation in ramp volumes.</td></tr>
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SI* (MODERN METRIC) CONVERSION FACTORS				
APPROXIMATE CONVERSIONS TO SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1,000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2,000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2,000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	2.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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LIST OF ABBREVIATIONS

A	incapacitating injury
AADT	annual average daily traffic
AASHTO	American Association of State Highway and Transportation Officials
AF	adjustment factor
ANSI	American National Standards Institute
B	non-incapacitating injury
C	possible injury
CD	compressed diamond
CLF	full cloverleaf
COV	coefficient of variation
CPM	crash prediction model
CURE	cumulative residual
DDI	diverging diamond interchange
DLT	displaced left-turn interchange
DOT	department of transportation
FHWA	Federal Highway Administration
FI	fatal injury
GIS	geographic information system
GPS	Global Positioning System
HSIS	Highway Safety Information System
HSM	*Highway Safety Manual*
IAR	Interchange Access Request
ID	identity
IJR	Interchange Justification Report
IOR	Interchange Operations Report
K	fatal
KA	fatal and severe injury
KABC	fatal and injury crash categories
KABCO	injury classification scale
LRS	linear referencing system
Parclo	partial cloverleaf
PDO	property damage only
PM	parallel merge
SDF	severity distribution function
SPDI	single-point diamond interchange
SPF	safety performance function
SPUI	single-point urban interchange
SR	single roundabout interchange
TDI	tight diamond interchange
VIEDA	variable introduction exploratory data analysis

CHAPTER 1. INTRODUCTION

BACKGROUND

In 2014, the American Association of State Highway and Transportation Officials (AASHTO) published a supplement to the *Highway Safety Manual* (HSM) containing crash prediction models (CPMs) for freeway segments, ramp segments, and ramp terminals.(1) To predict crashes for these facilities, these models require detailed sets of inputs, down to specific design characteristics for features such as lane width, shoulder width, and clear zone. One potential use of the HSM freeway and interchange models is to predict the potential safety performance effects of new interchanges or modifications to existing interchanges.

Documentation of potential safety performance impacts is required for the justification and documentation necessary to substantiate any proposed changes in access to the Interstate System.(2) Typically, Interchange Justification Reports (IJRs) are written early in the project planning and design process, with details generally consistent with conceptual design. As such, the design details required for using the HSM CPMs may not be known; only the general form of the interchanges is known. Without these details, using the HSM models to develop an accurate prediction of crash frequency and severity for the design is difficult. In addition, aggregating component-by-component (e.g., individual ramp and freeway segment) predictions may not fully capture the safety performance impacts when considering the project location as a whole.

To further explore and address an IJR application, the Federal Highway Administration (FHWA) sought to develop planning-level models and tools to predict crash frequency and severity for an existing or proposed interchange. These planning-level models will allow analysts to compare the potential safety performance effects of freeway access and interchange design decisions at the planning level. Model inputs were limited to details that are generally known at the planning and conceptual design level and are expected to affect crash frequency and severity (e.g., traffic volume, number and forms of ramps, number and types of ramp terminals, land use, and mainline cross section).

Since most IJRs focus on a few specific interchange types, the scope of the planning-level models focuses on those that are most often considered. This research identified those interchange configurations that State agencies most commonly consider in IJRs (not just those that are selected), identified features related to safety performance that are typically assessed during the IJR review process, and developed a model for assessing those particular factors. The project team developed a spreadsheet implementation tool to support analysts using the methods described in this report. The spreadsheet tool directly implements the methods, geometric and operational characteristics, and parameters described in this report.

OBJECTIVE

The objective of the research study was to develop a planning-level safety assessment tool and interchange safety comparison process for FHWA and State department of transportation (DOT) use for IJR reviews. The safety assessment tool will allow agencies to quantify the safety performance of proposed designs against a base (or reference) condition.

PURPOSE AND STRUCTURE OF THE GUIDE

This report aims to document the process used to develop the planning-level predictive model, including factors considered and not included in the final application. The lessons learned from this project can inform planning-level predictive safety analysis and future improvements to the IJR process and predictive approach in the HSM.(3) This report is organized into the following seven chapters:

1. Introduction: Provides an overview of this research project and the purpose of this report.

2. Identify Interchange Types for Inclusion in This Study: Discusses the survey and identifies the interchange types reviewed for IJRs and recommended for further analysis.

3. Methodology for Predicting Safety Performance of Interchanges: Details the methodology used for developing the predictive models, the final crash frequency and severity models, and comparative application of the planning-level model relative to design-level models from the HSM.

4. Data Collection: Describes the data collection that supported the development of the safety prediction model and the methodology for predicting safety performance.

5. Predictive Model Development: Describes the development of the crash frequency prediction model, including a description of the planning-level predictive method for interchanges.

6. Severity Distribution Function (SDF) Development: Describes the development of SDFs, including a description of the planning-level factors associated with crash severity, given a crash has occurred.

7. Conclusions and Recommendations: Summarizes the report's findings and identifies future research possibilities.

CHAPTER 2. IDENTIFY INTERCHANGE TYPES FOR INCLUSION IN THIS STUDY

OBJECTIVE

This chapter describes the survey the project team administered to FHWA division offices to identify which interchange configurations are most often considered for IJR reviews. The survey aimed to identify interchange configurations that comprise at least 75 percent of those considered for inclusion in the predictive methodology developed in this research. The survey also solicited information on approximately how many interchanges of each configuration are constructed (and considered) annually to determine the potential availability of data for developing the predictive method. Further survey questions identified States for data collection and determined features of interest for safety prediction during the planning stage, both of which the project team refined in the data collection and analysis study design. Following is an overview of the survey responses and FHWA recommendations for interchange configurations for inclusion in the study. Appendix A includes the survey instrument.

SURVEY RESPONSE

The project team provided an electronic survey to all FHWA divisions based on the FHWA-provided list of those most likely to be involved with reviewing IJRs. In a dedicated followup effort, the project team obtained responses from FHWA divisions in 45 States; Puerto Rico; and Washington, DC.

Table 1 summarizes the results of survey questions 6 through 11. Responses to questions 1 through 5 are not included in this summary, as they simply collected contact information from the survey responder. Each question received responses that added up to 47 jurisdictions, except in cases where not everyone responded.

Table 1. Responses to questions 6 through 11.

Question	Response	Number of Responses*
6. How many IJRs does your State typically submit per year?	*0–5*	*35*
	6–10	8
	11–15	2
	16–20	0
	More than 20	1
7. Of those IJRs, how many involve system interchanges (interchanges providing access from one freeway to another)?	*0–2*	*39*
	3–5	6
	6–8	1
	More than 8	1
8. Approximately what percentage of service interchange IJRs (interchanges that provide access between a freeway and a nonfreeway road network) are for rural interchanges (as opposed to urban/suburban interchanges)?	*0–25*	*29*
	26–50	11
	51–75	2
	More than 75	5
9. Approximately how many service interchange IJRs involve a new interchange?	*0–5*	*46*
	6–10	1
	11–15	0
	More than 15	0
10. Approximately how many service interchange IJRs involve reconfiguring an existing interchange?	*0–5*	*40*
	6–10	6
	11–15	0
	More than 15	1
11. Approximately how many service interchange IJRs per year involve minor changes (such as modifying ramp terminal access control, adding lanes to an existing ramp, or relocating a ramp terminal to a different roadway)?	*0–5*	*40*
	6–10	3
	11–15	2
	16–20	1
	More than 20	0
	No response	1

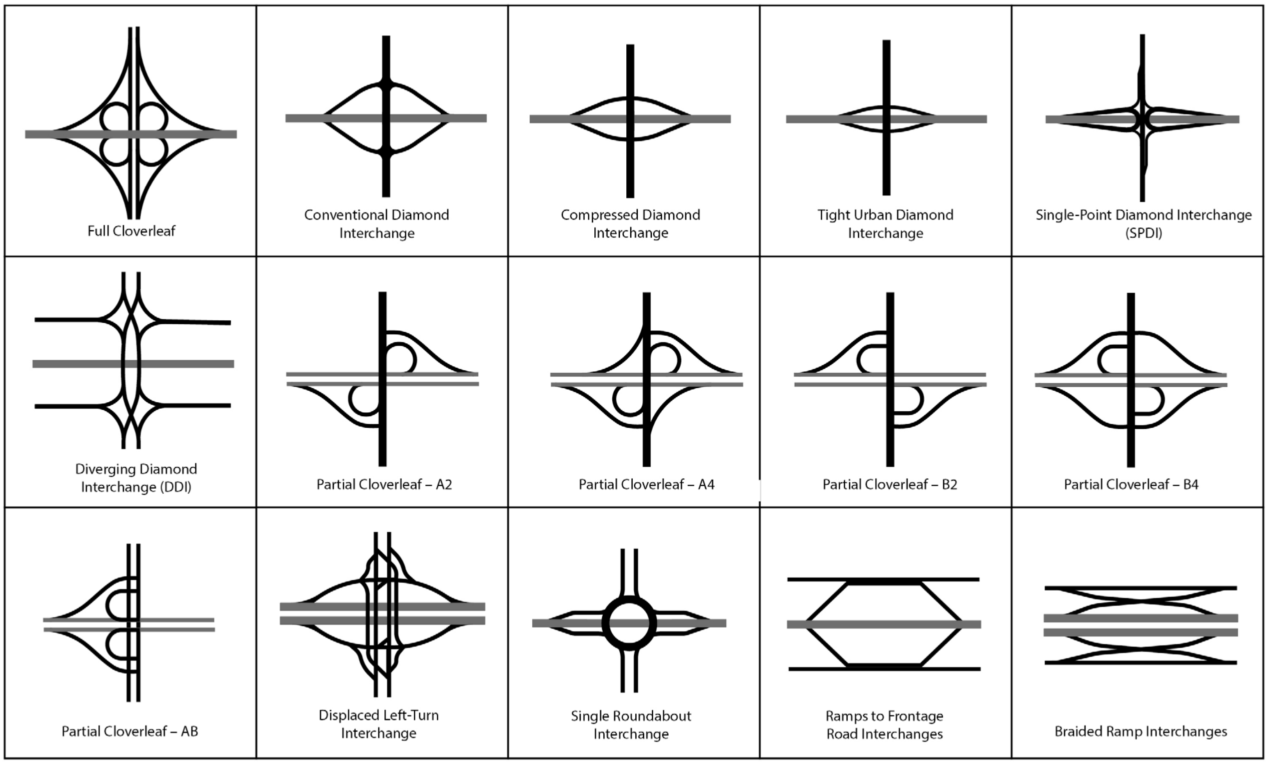
*Categories with the highest number of responses are italicized.

The focus of this project, and predictive modeling, was on service interchanges. The results indicate that most divisions receive fewer than five IJRs per year, with most of those focused on service interchange projects. However, some larger States receive a substantial number of IJRs per year, with one agency reporting more than 20 IJRs per year.

Predictably, question 8 responses indicate that the majority of IJRs focus on urban applications, particularly when weighted by the number of IJRs received (i.e., divisions with a higher percentage of rural IJRs tend to receive fewer IJRs). Additionally, more rural States, such as Wyoming, had a higher percentage of rural IJRs.

The majority of respondents indicated reviewing 0 to 5 new interchange IJRs per year in question 9 and provided similar responses for reconfiguration of existing interchanges in question 10; however, 6 respondents indicated 6 to 10 interchange reconfigurations per year, and 1 respondent indicated more than 15 per year. The respondents provided similar responses for question 11 for minor interchange modifications.

For question 12, the project team provided respondents with 15 potential interchange configurations and asked how often each configuration was considered by each agency annually. Figure 1 provides an overview of the interchange configurations included in the survey.



Source: FHWA.

Figure 1. Graphic. Interchange configurations considered in the survey.

The questionnaire included the following response categories for how often an interchange configuration is considered by each agency:

1. Uncommon or rarely considered.
2. Sometimes considered but rarely selected.
3. Commonly considered and selected zero to two times per year.
4. Commonly considered and selected three to five times per year.
5. Commonly considered and selected six to eight times per year.
6. Commonly considered and selected more than eight times per year.
7. No response recorded.

Table 2 provides the results for question 12, which asked respondents to estimate the frequency of consideration and selection of various interchange types. Note that the number in the response column in table 2 corresponds to the response category defined in the previous list. Responses 1 and 2 indicate the interchange configuration may or may not be considered but is not implemented by the agency. Responses 3 through 6 indicate the interchange configuration is considered and implemented. Response 7 provides a space for divisions that did not provide a response for their particular agency.

Looking at the responses in table 2, full cloverleaf (CLF), displaced left turn (DLT), single roundabout (SR), ramp to frontage road, and braided ramp interchanges were not common configurations implemented by many agencies. A follow-on question (question 13) asked respondents to identify any potential inclusions that were missing from the list in table 2. Respondents indicated double roundabout, or dogbone, interchanges were missing. However, the project team considered roundabouts a traffic control approach for ramp terminals, and too few agencies are currently using double roundabout interchanges; therefore, they were not prioritized for inclusion as a separate interchange type. Additional responses focused on system interchanges, partial interchanges, and split interchanges.

Table 2. Number of responses to frequency of consideration and selection of various interchange types.

Response	CLF	D	CD	TDI	SPDI	DDI	A	B	AB	DLT	SR	RF	BR
1	*24*	6	8	6	9	8	9	13	*16*	*36*	*26*	*22*	*23*
2	13	3	8	8	14	5	12	13	13	4	6	9	10
3	6	*24*	*22*	*25*	*19*	*25*	*20*	*15*	13	0	6	9	7
4	0	9	1	1	0	4	0	0	0	0	1	0	0
5	0	1	1	0	0	0	0	0	0	0	1	0	0
6	0	0	0	0	0	0	0	0	0	0	0	0	0
7	4	4	7	7	5	5	6	6	5	7	7	7	7

*Categories with the highest number of responses are italicized.
D = diamond; CD = compressed diamond; TDI = tight diamond interchange; SPDI = single-point diamond interchange; DDI = diverging diamond interchanges; A = partial cloverleaf (parclo) A2 or A4; B = parclo B2 or B4; AB = parclo AB2 or AB4; RF = ramp to frontage road; BR = braided ramp.

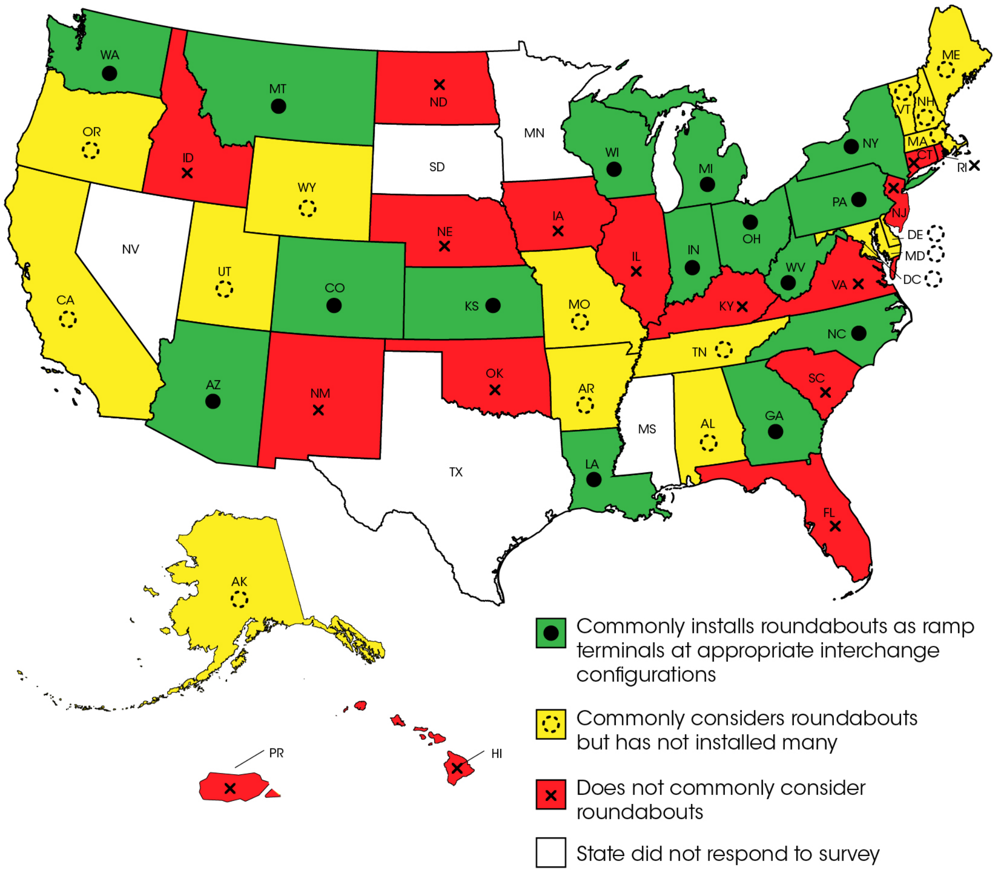
To identify potential States for data collection, the project team asked follow-on questions about which agencies consider and implement alternative interchanges, including single-point diamond interchanges (SPDIs), diverging diamond interchanges (DDI), DLT interchanges, and SR interchanges. Most responding divisions indicated their agencies commonly consider and install alternative interchanges, except most New England States, Maryland, Nebraska, New Jersey, and Wyoming. Division representatives also indicated Alaska and Hawaii do not consider and install alternative interchanges as of the time of this survey.

Table 3 further indicates the number of agencies commonly considering or installing roundabouts at ramp terminals. Figure 2 shows agencies that typically install roundabouts at terminals and those that commonly consider installing them. The responses indicate an even divide across the country for agencies installing roundabouts, only considering roundabouts, or not installing roundabouts at ramp terminals.

Table 3. Responses to whether agencies install roundabouts at terminals.

Response	Number of Responses*
Does not commonly consider roundabouts	*16*
Commonly considers roundabouts but has not installed many	*16*
Commonly installs roundabouts as ramp terminals at appropriate interchange configurations	15

*Categories with the highest number of responses are italicized.



Source: FHWA.

Figure 2. Graphic. Agencies installing roundabouts at ramp terminals.

DISCUSSION

Table 4 summarizes the approximate number of IJRs by interchange configuration per year. The table includes a column for agencies considering, but not installing, the configuration. For this scenario, the project team assigned 0.25 points per agency to provide weight for agencies considering but not installing. Each of the other columns is weighted to the average number of years for the category selected (e.g., each interchange in the zero to two per year category receives a weight of one and interchanges in the three to five per year category receive a score of four). The total score comes to 357 interchanges per year when including considered interchanges and 276 for constructed interchanges. Additional comments provided by State agencies regarding their respective IJRs are listed in appendix B.

Table 4. Estimated number of IJRs by interchange configuration per year.

Interchange Configuration	Considered Only	0–2 Per Year	3–5 Per Year	6–8 Per Year	9+ Per Year	Total Score	Total Score Constructed Only
Score weight	0.25	1	4	7	9	—	—
CLF	37	6	0	0	0	15.25	6
Conventional diamond	9	24	9	1	0	69.25	67
CD	16	22	1	1	0	37	33
TDI	14	25	1	0	0	32.5	29
SPDI	23	19	0	0	0	24.75	19
DDI	13	25	4	0	0	44.25	41
Parclo A	21	20	0	0	0	25.25	20
Parclo B	26	15	0	0	0	21.5	15
Parclo AB	29	13	0	0	0	20.25	13
DLT	40	0	0	0	0	10	0
SR	32	6	1	1	0	25	17
Ramps to frontage road	31	9	0	0	0	16.75	9
Braided ramp	33	7	0	0	0	15.25	7
Total	*324*	*191*	*16*	*3*	*0*	*357*	*276*

—No data.

Table 5 ranks interchange configurations by individual score (based on installations per year). The table indicates the interchange configurations that are expected to comprise at least 75 percent of IJRs. Conventional diamond interchanges received the highest score, and partial cloverleaf (parclo) AB interchanges received the lowest score for those in table 5. When excluding parclo AB interchanges, the list still includes 78 percent of those interchanges considered and 87 percent of interchanges constructed; however, the project team felt parclo AB interchanges are consistent with parclo A and B interchanges and would be worth including in the dataset moving forward.

Table 5. Interchange configuration ranking by score.

Interchange Configuration	Total Score	Constructed Score
Conventional diamond	69.25	67
DDI	44.25	41
CD	37	33
TDI	32.5	29
Parclo A	25.25	20
SR	25	17
SPDI	24.75	19
Parclo B	21.5	15
Parclo AB	20.25	13
Total	299.75	254
Percentage	84	92

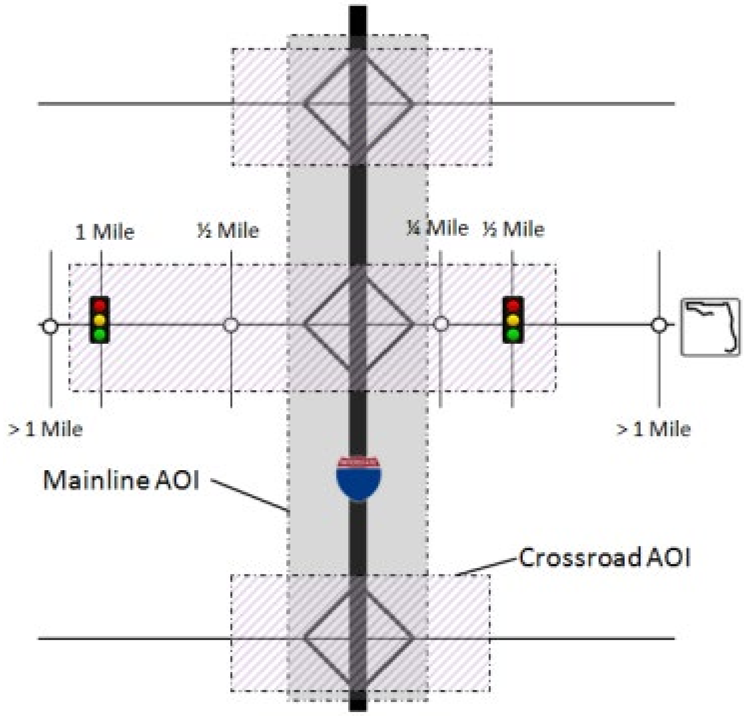
CHAPTER 3. METHODOLOGY FOR PREDICTING SAFETY PERFORMANCE OF INTERCHANGES

INTRODUCTION

This chapter describes the framework for developing planning-level CPMs for the more common interchange configurations identified in the survey results described in chapter 2. First, this chapter provides an overview of the interchange influence and interchange areas used to define the analysis coverage and applicable area for crash prediction. Next, this chapter presents the approach to predicting crash frequency using the interchange as the base unit of analysis. Finally, this chapter discusses the approach to predicting crash severity using a fatal or injury crash as the base unit of analysis.

INTERCHANGE INFLUENCE AREA

The general study area, or area of influence, necessary for conducting safety and operational analysis for interchange access improvements can vary substantially in size and scope. While access improvements should be considered on a case-by-case basis, the study area should generally include the components of the highway network adjacent to the interchange of interest that could impact, or be impacted by, the presence of the interchange(s) of interest. This highway network includes, at a minimum, the upstream and downstream interchanges and to the first adjacent signals on the crossroad.(2,4) The area of influence should be increased if safety and operational influences are likely to exist beyond those noted to capture the impacts to the broader system (e.g., impacts of traffic pattern changes on the crossroad network).(2) Figure 3 provides a sample influence area along a freeway mainline and crossroad for Florida DOT Interchange Access Requests (IARs).(5) The study area, in this case, includes the upstream and downstream existing interchanges.



© 2020 Florida DOT.
AOI = area of influence.

Figure 3. Graphic. Area of influence for proposed interchange analysis in Florida.[(5)]

INTERCHANGE AREA

Due to the case-by-case nature of the influence area, the project team determined a planning-level predictive method based on the influence area was impractical. Therefore, the project team proposed the predictive method focus on the interchange area.

Given the turbulence caused in the traffic stream, it is still difficult to isolate the effects of a single interchange by determining the interchange area when using a one-size-fits-all approach. Isolating the effects can be especially challenging when comparing interchange alternatives for one location—some interchange configurations are inherently longer than others, particularly with respect to the dimension of the interchange along the crossroad. A diamond or parclo interchange with widely spaced ramp terminals spans a much longer length of the crossroad than a smaller interchange such as a tight diamond interchange (TDI) or an SPDI. As such, it was important to develop clear but flexible criteria to define the interchange area, adapting to the specific model application.

Criteria for defining the interchange area are dependent on the location/geometry of the gores and speed change lanes (acceleration and deceleration lanes) on the mainline and the ramp terminal(s) on the crossroad. When applying the predictive models for safety comparisons, users should determine the maximum possible interchange area dimensions among the alternatives considered and apply these dimensions to all alternatives. For alternative interchanges with a smaller footprint, users would apply the HSM predictive models to account for differences in interchange area dimensions among the alternatives. Using HSM predictive models is similar to evaluating safety for a curve that has been reconfigured to have a larger radius. The curve is lengthened to accommodate the larger radius. If the analyst does not account for the reduction in tangents on each side of the curve, then the new curve may have more predicted crashes than the old curve due to the increased overall length. For a fair comparison, analysts should account for the safety effect of reducing the tangents.

The project team considered the following two methods for evaluating the interchange area:

1. Use the farthest end of the taper on each side of the crossroad to define freeway influence. In cases where a lane add or drop is used (i.e., no taper exists), then a length of 1,500 ft from the farthest painted gore from the crossroad on each side would be used. For cases of closely spaced interchanges, this length would be reduced to the midpoint between the adjacent interchanges (measured from the nearest painted gores between adjacent interchanges).

2. Apply the American National Standards Institute (ANSI) definition of an interchange area.[6] Under this definition, the interchange area extends 100 ft beyond the physical gore point furthest from the crossroad. It is also defined on the crossroad as extending 100 ft beyond the outermost curb return for each terminal.

The difference is that method 1 provides an opportunity to consider the entire length of a speed change lane when a taper is present but may reduce the influence area when shorter speed change lanes are present—suggesting safety is only impacted upstream/downstream when longer speed change lanes are present. This assumption is not consistent with the findings of Bonneson et al., which suggested that lane changing impacts safety up to one-half mi upstream or downstream of a gore; but the effect is reduced as distance increases.[7]

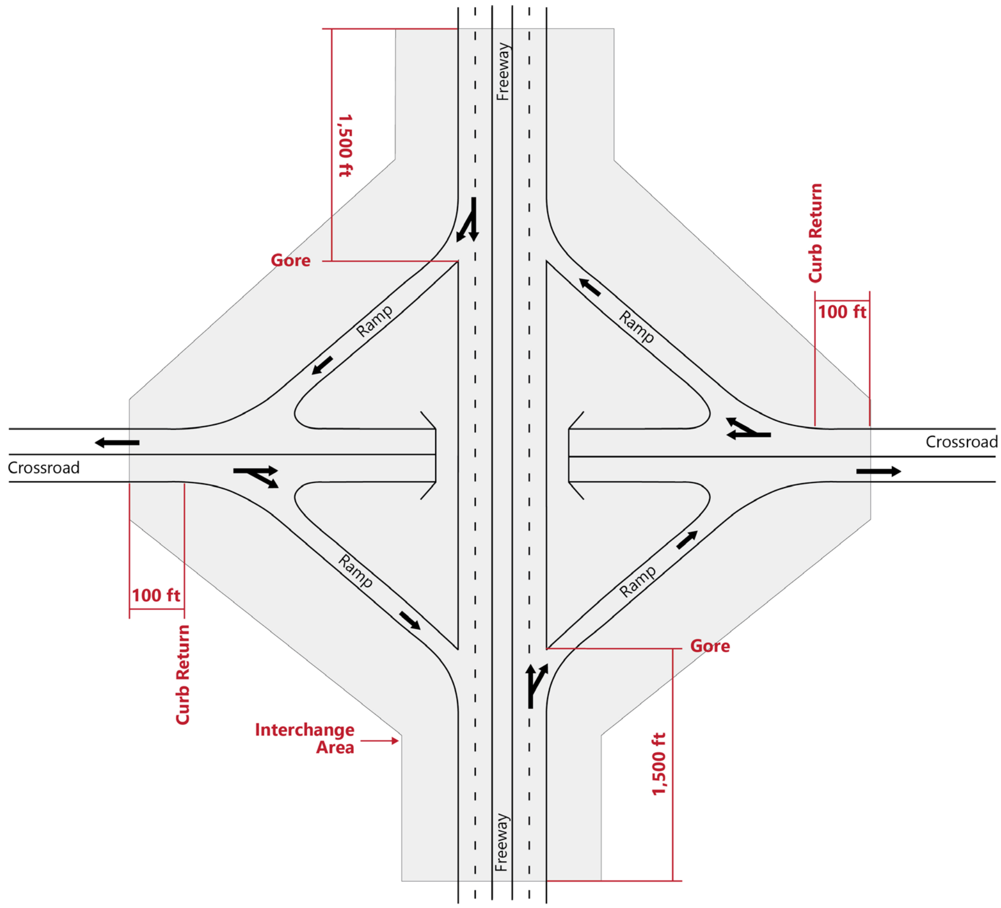
The project team opted to use a modified form of the ANSI definition, which includes 1,500 ft upstream and downstream of the painted gore. This definition is consistent with the HSM approach for painted gores and is consistent with planning-level safety analysis, which commonly defines interchange influence as within 1,500 ft of the crossroad. This approach is advantageous for several reasons:

- The first approach is dependent on the location of tapers for interchange speed change lanes. At the planning stage, the rough location of gores may be known, but it is likely that acceleration and deceleration lanes are not yet defined, and assumptions will be needed on the location of the taper. Using a uniform 1,500-ft influence value eliminates the need to make assumptions regarding taper locations.

- The approach is standardized for all interchange configurations and for entrance and exit conditions (e.g., lane add/drop). The project team included data identifying the influence of adjacent facilities on safety performance, negating the need to address more formally those in the definition of the interchange area.

- The approach provides more uniformity across interchange configurations of different sizes and scopes as well as for older interchanges with more compressed features relative to newer interchanges in western States where space may be of less consideration.

Figure 4 provides a graphical representation of the interchange area for a diamond interchange. For cases where interchanges are closely spaced, the midpoint between the interchanges is used in lieu of 1,500 ft.



Source: FHWA.

Figure 4. Graphic. Study interchange area definition.[8]

The crossroad interchange area is defined by characteristics on the outside legs of the crossroad ramp terminal(s). For a crossroad ramp terminal, the crossroad interchange area is defined as 100 ft outside of the gore or curb return of the outermost ramp connection for each terminal. This definition is based on the description of an interchange crash from ANSI D16.1-2007.[6] Figure 5 shows how this measurement can be determined for an SPDI, while figure 6 provides an aerial markup of the identification on a compressed diamond (CD) interchange.[9] The total crossroad distance was measured and explored as a potential predictor variable in the CPM. Additionally, the project team captured the distance to adjacent features related to safety as outlined in the data collection section.

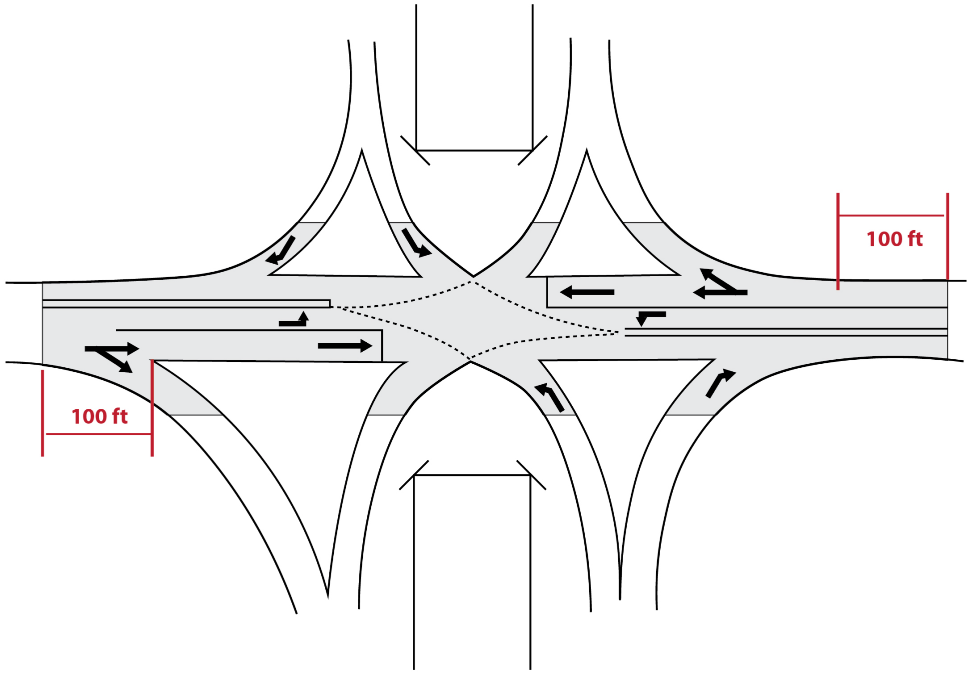


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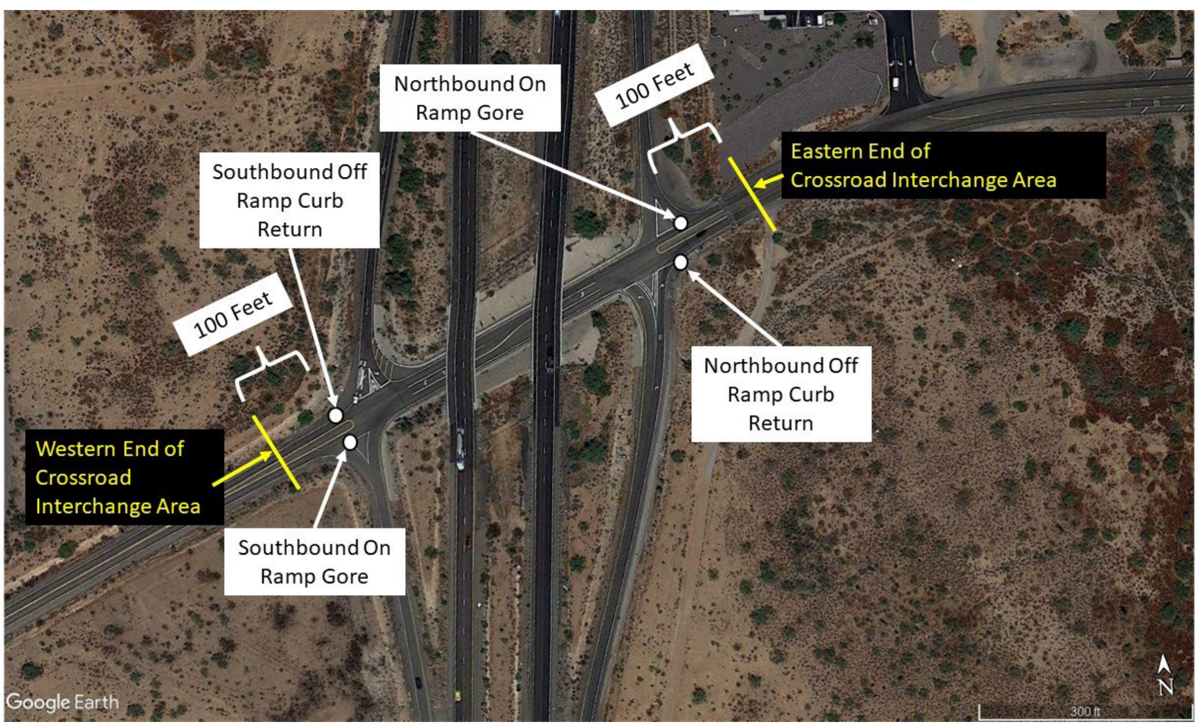
Figure 5. Graphic. Example crossroad interchange area for an SPDI.[(9)]



Original map: © 2021 Google® Earth™. Modifications by FHWA to show the example ramps (see Acknowledgments section).

Figure 6. Photo. Example crossroad interchange area.[(10)]

CRASH FREQUENCY ANALYSIS METHODOLOGY

For this research, the project team developed a predictive method for planning-level safety prediction to compare interchange configuration safety performance. The methodology is consistent with the predictive methods included in the HSM for project-level analysis, particularly freeway segment, ramp, and ramp terminal project-level analysis.

Project-level safety performance functions (SPFs) are part of an overall CPM in Part C of the HSM.[(3)] A CPM estimates the predicted average crash frequency of a specific type of site (e.g., interchange) with specific geometric design elements and traffic control features. Each CPM in the HSM has the form shown in figure 7.

$$N_p = C \times N_{SPF} \times (AF_1 \times \ldots \times AF_n)$$

Figure 7. Equation. HSM CPM.

Where:

N_p = predicted average crash frequency, crashes/year.
C = local calibration factor.
N_{SPF} = predicted crash frequency for a site with base conditions.
AF_i = adjustment factor (AF) for geometric design element or traffic control feature i (i=1 to n).
n = total number of AFs.

Each CPM includes an SPF, one or more AFs, and a local calibration factor. The SPF predicts the crash frequency N_{SPF} for a site having characteristics that match a specified set of "base conditions." These conditions describe the typical site's design elements and control features (e.g., no left-turn lanes at terminals). The set of AFs is used to adjust N_{SPF} such that the CPM can provide reliable estimates of the predicted crash frequency N_p for sites that do not match all base conditions. In the case of planning-level models for interchange configuration, AFs focus more on the types of features present (e.g., numbers of lanes by type or traffic control at terminals) rather than specific geometric conditions (e.g., outside shoulder width), as they are generally not known at this stage. The calibration factor simply adjusts the base SPF estimate up or down to adjust predictions to match observed crash frequency for a local jurisdiction.

The project team considered developing separate SPFs using the following characteristics:

- Interchange configuration.
- Crash type—Multiple-vehicle versus single-vehicle crashes.
- Crash severity—Fatal and injury (KABC on the KABCO injury classification scale, where K represents fatal, A incapacitating injury, B non-incapacitating injury, and C possible injury crashes) versus property damage only (PDO) crashes (O on the KABCO scale).

As described in chapter 4, sample sizes were not sufficient to estimate SPFs reliably for individual interchange configurations or by crash type. Therefore, the SPFs focus on KABC crash frequency and PDO crash frequency separately.

The predictive method for interchange areas provides crash frequency and severity by type for consistent application as necessary. The predictive models are consistent with the project-level predictive models in HSM Part C but function as planning-level inputs and analysis.(3) This aspect of the research project is important as the predictive models for interchanges will likely need to be combined with other HSM predictive models to provide evaluations for the entire influence area for IJRs.

Figure 8 serves as the foundation for the interchange crash frequency CPM.

$$N_{y,cs} = y \times L \times \exp\left(b_0 + b_1 \ln[AADT_{fr}] + b_2 \ln[AADT_{xs}] + b_3 \ln[AADT_r] + \sum b_i I_i\right) \times (AF_1 \times \ldots \times AF_n)$$

Figure 8. Equation. Interchange crash frequency CPM.

Where:

$N_{y,cs}$ = predicted average crash frequency for crash severity (KABC or PDO) and for year y.
y = time interval for reported crashes (i.e., evaluation period) per year.
L = segment length (freeway, crossroad, and ramp lengths are considered).
b_0 = regression coefficient for constant.
b_1, b_2, b_3 = regression coefficients for freeway, crossroad, and ramp annual average daily traffic (AADT), respectively.
$AADT_{fr}$ = freeway AADT.
$AADT_{xs}$ = crossroad AADT.
$AADT_r$ = ramp AADT (explored as total ramp AADT and AADT by ramp type).
b_i = regression coefficient for jurisdiction (i.e., State) i.
I_i = indicator variable for State i (=1.0 if data corresponds to site i; otherwise 0).

For the full model, the project team inferred AFs from the multiple-variable regression model. The project team considered AFs for planning-level features (chapter 4), such as distance to adjacent interchange ramp gore, where the safety impact is consistent across interchange configurations. Additionally, the project team considered interactions to identify where and when a safety effect may differ from one interchange configuration to another. Chapter 4 provides specific interchange components considered and included in final crash frequency models.

The project team used a generalized database to estimate the planning-level CPMs. The new CPMs were coded as a multiple-variable regression model, and regression analysis was used to compute the regression coefficients. The AFs developed for the predictive method were inferred from the multiple-variable model such that they can be used together. Separate CPMs were estimated for fatal injury (FI) and PDO crash frequency.

For crash frequency modeling, count models are traditionally used to quantify the relationship between crash frequency and traffic volumes, design elements, and traffic control features. Negative binomial regression has commonly been applied to account for the overdispersion inherently found in crash data. The overdispersion parameter estimated from the modeling process is used in the development of the weight factor in the empirical Bayes analysis method.

Recently, researchers have applied more sophisticated versions of count models to account for temporal and spatial correlations. Depending on the assumption of the correlation between unobserved effects and right-hand side variables, fixed- and random-effects models have been applied to account for temporal and spatial correlations. As models become more sophisticated, bias and inconsistency are generally lessened, improving the transferability of the model. However, more sophisticated models can be more time consuming to estimate, limiting the number of models that can be estimated during the modeling process. Additionally, more sophisticated models can prove to be more difficult to reach convergence with a larger number of predictor variables, limiting the number of geometric and operational features that can be included in the model specification.

The project team considered fixed-effects, random-effects, and mixed-effects models to account for unobserved correlations inherent in the data. The project team focused on negative binomial count models, as is the current state of the practice.

The project team used the variable introduction exploratory data analysis (VIEDA) approach, which differs from the typical approaches (forward and backward selection) used for regression modeling.[(11)] The intent of VIEDA is to answer the following two questions:

1. Is the variable related to safety?
2. What is the proper functional form the variable should take in the CPM?

The VIEDA approach is a two-step process. The first step is to develop a foundational model that includes key variables that define the facilities and exposure (e.g., AADTs, component lengths, interchange configuration). The foundational model is then used to predict crashes for each of the interchanges included in the model. In the second step, the observed crashes and fitted crashes are compared for each variable considered for inclusion in the model. If the ratio of observed crashes to fitted crashes exhibits a regular relationship with the variable considered for addition to the model, the variable is introduced, and the relationship exhibited is used to identify the proper functional form of the new variable. This methodology helps to identify the best form for continuous variables, as well as helps to identify potential dividing points for creating indicator variables. Once the variable is introduced to the model, the estimated parameters and associated standard errors are examined for the following:

- Is the direction of effect (i.e., expected decrease or increase in crashes) in accord with expectations?
- Does the magnitude of effect seem reasonable?
- Are the parameters of the model estimated with statistical significance?
- Does the estimated overdispersion parameter improve?

The project team asked these questions whenever a new variable was introduced to the model. In this manner, the project team gained an intimate understanding of the data and potential relationships. The process was iterative and allowed the team to look at potential holes and errors in the data as well as correlations impacting the development and refinement of the overall CPM.

CRASH SEVERITY ANALYSIS METHODOLOGY

While the CPM predicts crash frequency (by combined injury severity categories and crash types), the project team developed SDFs to predict the proportion of K, A, B, and C crash severity categories. The probability of each severity category is predicted as a function of traffic volume, geometry, and other interchange characteristics. The proportion is multiplied by the predicted crash frequency to obtain an estimate of the crash frequency for the corresponding severity category.

This approach was shown by Avelar, Dixon, and Ashraf to be reliable for predicting crash frequency for individual severity levels, as opposed to developing crash severity-specific SPFs.(12) This approach is consistent with the project-level SPFs for freeways, ramps, and ramp terminals found in the HSM supplement.(1)

The project team used a multinomial logit model to estimate SDFs for the KABC CPM. The multinomial logit model allows for some variables to be constrained to have the same effect on each severity level while allowing other variables to have a differentiated effect among levels. For SDFs, the database was restructured such that the observed unit was the crash instead of the interchange. The FI crashes CPM can be combined with the SDF to estimate the number of crashes of different severity levels.

CRASH PREDICTION GOODNESS-OF-FIT MEASURES

The project team assessed the performance of the individual crash frequency models and compared alternative models to determine the most appropriate predictive models. The following are appropriate goodness-of-fit measures the project team used to assess the performance of new predictive models:

- Mean absolute deviation: Measures the average magnitude of variability of prediction. Smaller values are preferred to larger values in comparing two or more competing models.

- Modified R^2: Measures the amount of systematic variation explained by the CPM. Larger values indicate a better fit for the data in comparing two or more competing models.

- Dispersion parameter: Measures the amount of variance in the CPM. All else being equal, a CPM with less dispersion is preferred (i.e., a smaller value is preferred to a model with more dispersion).

- Cumulative residual (CURE) plots: Figure 9 illustrates CUREs (observed minus predicted crashes) against a variable of interest sorted in ascending order (e.g., freeway traffic volume). CURE plots such as the one shown help to identify the following concerns:

    - Long trends: Trends in the CURE plot (increasing or decreasing) indicate regions of bias.

    - Percent exceeding the confidence limits: CUREs outside the confidence limits indicate a poor fit over that range in the variable. CUREs frequently outside the confidence limits indicate notable bias in the model. A reasonable upper threshold for the percent of the CURE plot exceeding the 95-percent limits is 5 percent.

- Vertical changes: Large vertical changes in the CURE plot are potential indicators of outliers in the dataset.

The project team used goodness-of-fit measures to evaluate alternative CPMs developed using the methods described. Additionally, the project team used CURE plots to assess residuals to determine the overall model fit to observed data.

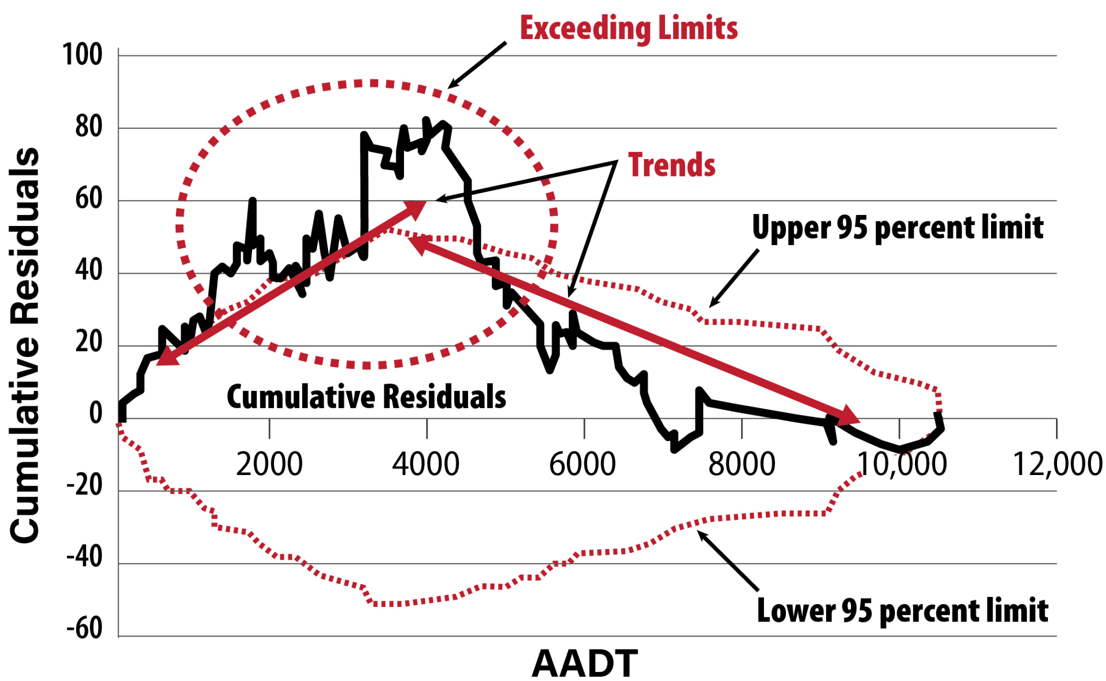


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Figure 9. Graph. Example CURE plot.[13]

CHAPTER 4. DATA COLLECTION

INTRODUCTION

This chapter describes this project's overall data collection process and the data integration effort to develop a combined project database for CPM estimation. An overview of the interchange configurations, crossroad ramp terminal configurations, ramp types, data elements collected, data collection methods, and data quality measures that the project team employed are included. This chapter also details the data integration approaches for developing the final generalized database and concludes with sample size details.

INTERCHANGE CONFIGURATIONS STUDIED

After surveying the FHWA division offices (results provided in chapter 2) and holding discussions with the project panel, the project team identified the following interchange configurations for inclusion in this study:

- Conventional diamond.
- CD.
- TDI.
- DDI.
- SPDI.
- Parclo type A2 and A4.
- Parclo type B2 and B4.
- Parclo type AB2 and AB4.

The survey respondents also indicated that SR interchanges are commonly considered and installed as service interchanges; however, the project team uncovered that when questionnaire respondents mentioned SR interchanges, they were usually referring to diamond interchanges with roundabouts at the crossroad ramp terminals.

To simplify the questionnaire, the project team had not differentiated between types of parclo configurations when surveying the FHWA division offices. However, for the study design, data collection, and analysis, the project team focused on parclo configurations that are most commonly considered and installed (A2, A4, B2, B4, AB2, and AB4) and developed models with the intent of capturing any safety performance differences between these parclo types. Due to the sheer number of possible combinations, the project team did not include other parclo configurations in this research.

Table 6 provides descriptions of the selected interchange configurations for inclusion in the predictive method. Additionally, figure 10 through figure 20 provide examples of those interchange configurations for reference.

Table 6. Descriptions of interchange configurations.

Interchange Configuration	General Description	Example Image
Conventional diamond	A traditional diamond interchange includes relatively straight entrance and exit ramps leading from the mainline to the crossroad. Two ramp terminals are spaced at least 800 ft apart.	Figure 10
CD	A CD interchange is a diamond with two ramp terminals spaced 400–800 ft apart.[7]	Figure 11
TDI	A TDI is a diamond interchange with signalized ramp terminals spaced 200–400 ft apart.[7,9] The signals on the two ramp terminals act in a coordinated effort, and turn lanes are created prior to the ramp terminal. No turn lanes are created between the ramp terminals.	Figure 12
DDI	A DDI is a diamond interchange that includes traffic crossovers on the crossroad, so turning movements to and from the ramps do not conflict with through traffic.	Figure 13
SPDI	This interchange is a variation of a diamond interchange in which all ramps meet at a single signalized intersection with the crossroad.[9]	Figure 14
Parclo A	In parclo A interchanges, the loop entrance ramp and diamond exit ramps are placed ahead of the crossroad. Parclo A4 also includes diamond entrance ramps beyond the crossroad.	Figure 15 (A2) Figure 16 (A4)
Parclo B	In parclo B interchanges, the loop exit ramps and diamond exit ramps occur beyond the crossroad. Parclo B4 also includes diamond exit ramps ahead of the crossroad.	Figure 17 (B2) Figure 18 (B4)
Parclo AB	A parclo AB interchange includes a combination of parclo A and parclo B features; typically, one direction includes the parclo A ramp orientations, and the other direction includes parclo B ramp orientations.	Figure 19 (AB2) Figure 20 (AB4)

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Figure 10. Photo. Aerial image of a conventional diamond interchange.[10]

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Figure 11. Photo. Aerial image of a CD interchange.[10]

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Figure 12. Photo. Aerial image of a TDI.[(10)]

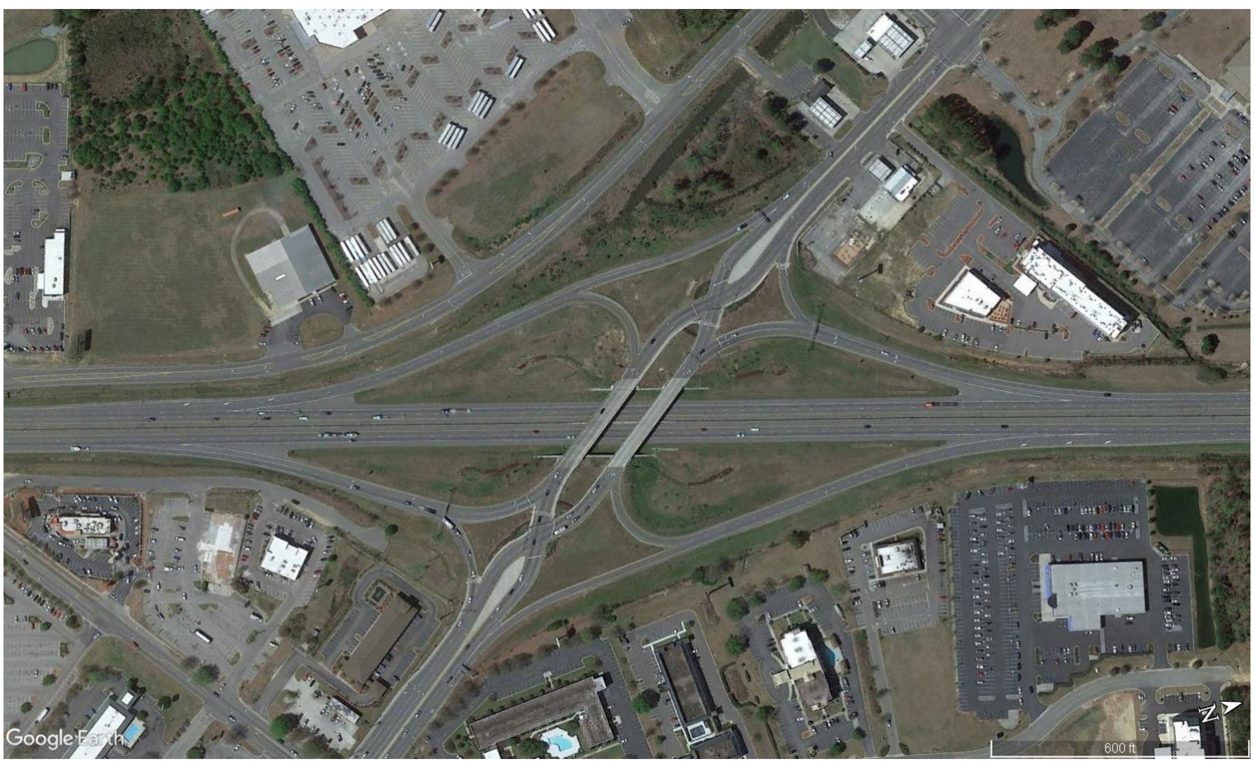

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Figure 13. Photo. Aerial image of a DDI.[(10)]

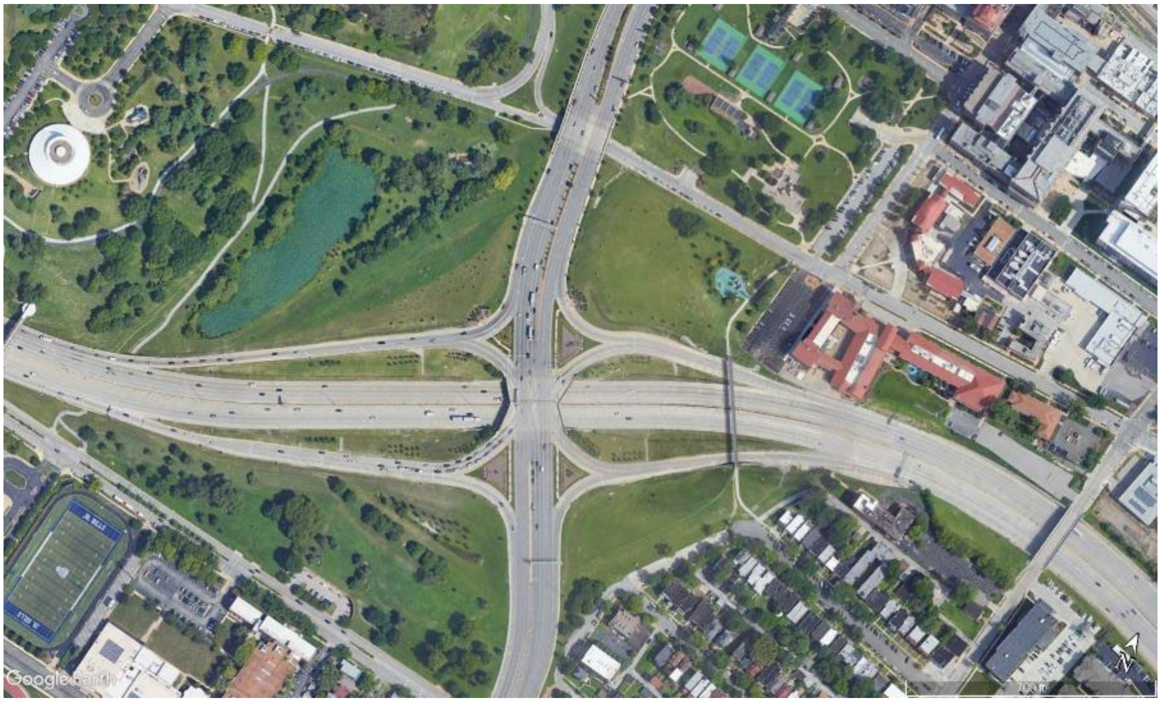

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Figure 14. Photo. Aerial image of an SPDI.[10]

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Figure 15. Photo. Aerial image of a parclo A2 interchange.[10]

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Figure 16. Photo. Aerial image of a parclo A4 interchange.[10]

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Figure 17. Photo. Aerial image of a parclo B2 interchange.[10]

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Figure 18. Photo. Aerial image of a parclo B4 interchange.[10]

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Figure 19. Photo. Aerial image of a parclo AB2 interchange.[10]

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Figure 20. Photo. Aerial image of a parclo AB4 interchange.[10]

CROSSROAD RAMP TERMINAL TYPES AND TRAFFIC CONTROL

While some crossroad ramp terminals are consistently used for certain interchange configurations, there can be some variance, such as a traditional versus roundabout configuration at a diamond interchange crossroad ramp terminal. Table 7 describes the crossroad ramp terminal configurations and related interchange configurations included in this study. Sketches in figure 21 through figure 29 provide examples of these crossroad ramp terminal configurations.

The project team also identified the traffic control for the ramp. The following forms of traffic control are valid for crossroad ramp terminals in this project:

- Signal control.
- Minor road stop control.
- All-way stop control.
- Roundabout control.
- Yield control.
- No control.

Note that right-turn movements may operate under different traffic control than the main terminal.

Table 7. Summary of crossroad ramp terminal configurations.

Ramp Terminal Configuration	Description	Potential Configurations	Example Image
D4	The intersection of a crossroad with an entrance ramp on one side of the road and an exit ramp on the other. Typical for diamond interchanges.	Diamond, CD	Figure 21
A4	The intersection of a crossroad with a diamond exit ramp and loop entrance ramp on one side and a diamond entrance ramp on the other. Typical for a parclo A4 interchange.	Parclo	Figure 22
B4	The intersection of a crossroad with a diamond entrance ramp and loop exit ramp on one side and a diamond exit ramp on the other. Typical for a parclo B4 interchange.	Parclo	Figure 23
A2	The intersection of a loop entrance ramp and an exit diamond ramp on the same side of the crossroad.	Parclo	Figure 24
B2	The intersection of a loop exit ramp and a diamond entrance ramp on the same side of the crossroad.	Parclo	Figure 25
TDI	Two closely spaced signalized ramp terminals that function in a coordinated manner.	TDI	Figure 26
SPDI	A single signalized ramp terminal with the crossroad and all ramps.	SPDI	Figure 27
DDI	A diamond interchange ramp terminal in which crossroad traffic temporarily flows in the opposite direction after crossing over at a signal.	DDI	Figure 28
Roundabout	A roundabout ramp terminal in which the circulating traffic has the right-of-way and traffic entering the roundabout enters under yield control.	Diamond, CD, parclo	Figure 29

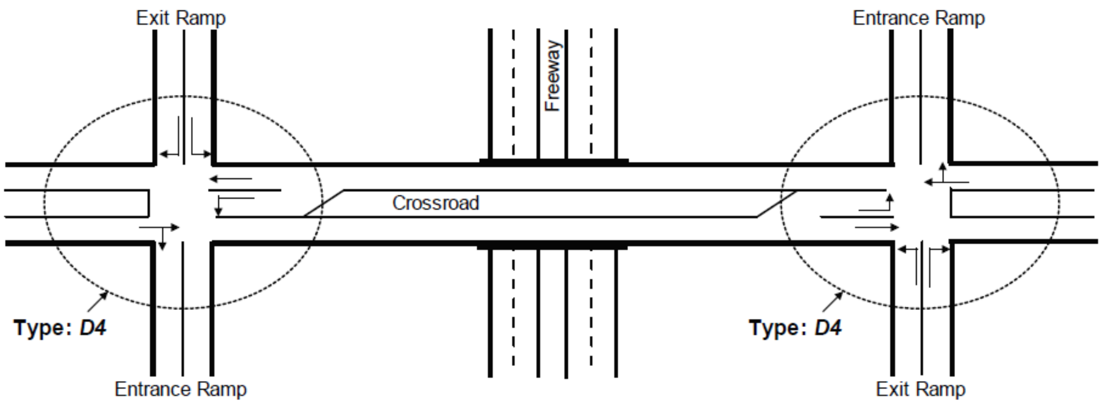

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Figure 21. Graphic. D4 ramp terminals.(1)

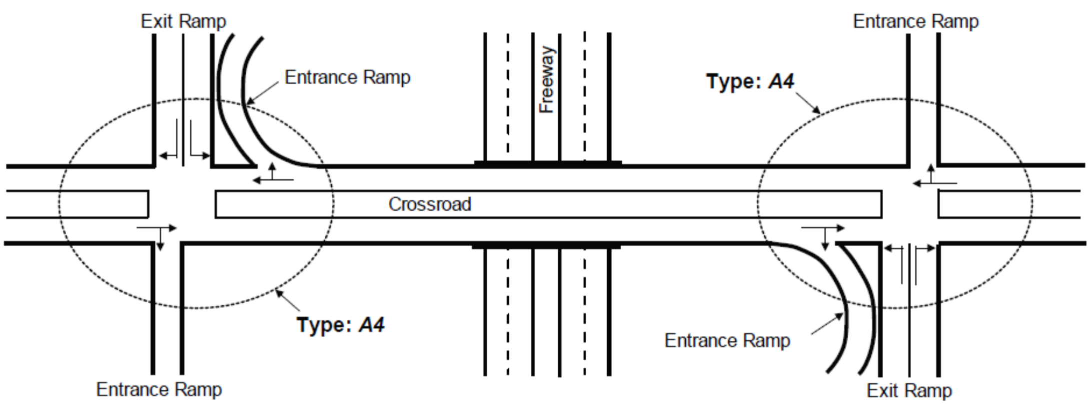

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Figure 22. Graphic. A4 ramp terminals.(1)

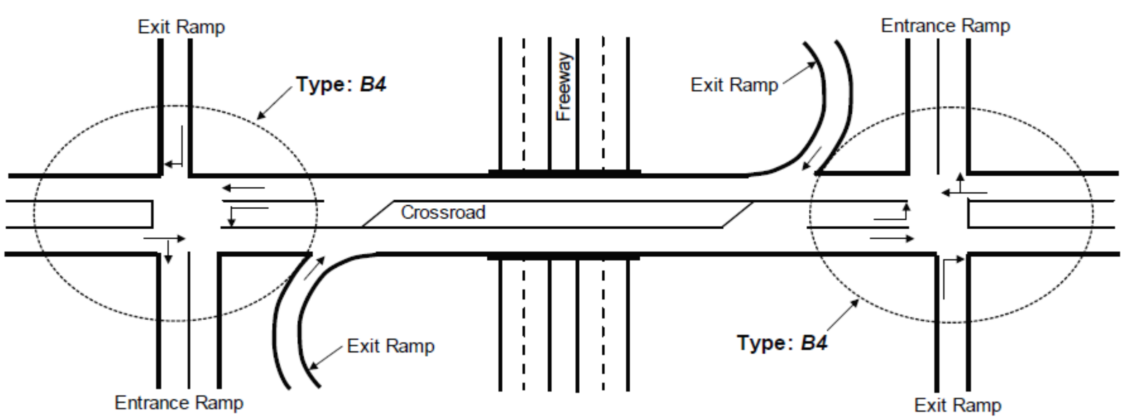

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Figure 23. Graphic. B4 ramp terminals.(1)

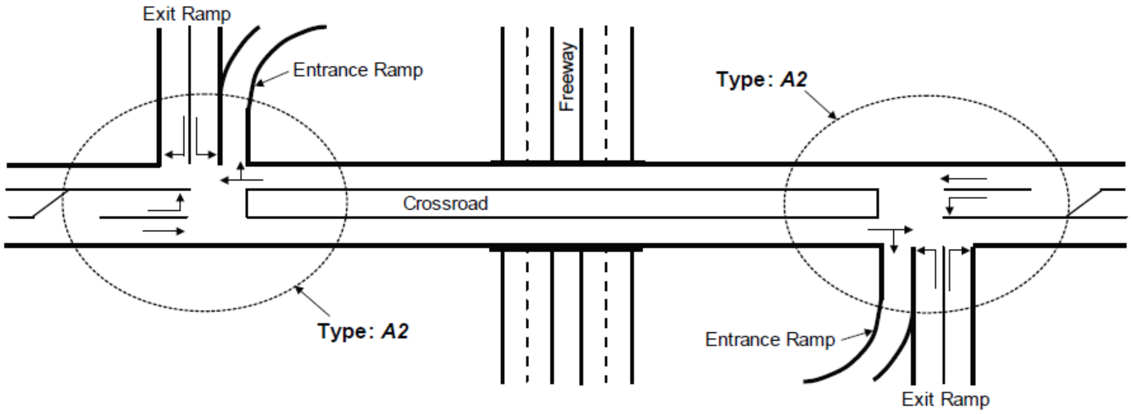

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Figure 24. Graphic. A2 ramp terminals.[1]

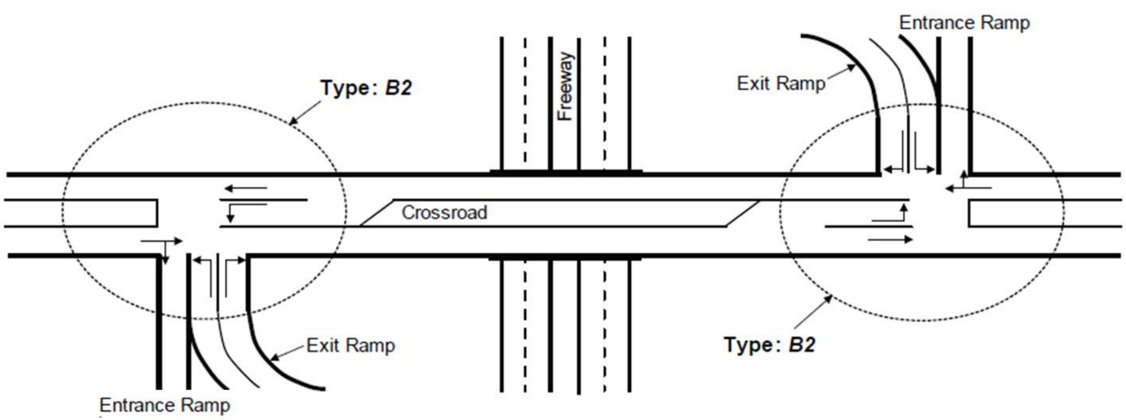

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Figure 25. Graphic. B2 ramp terminals.[1]

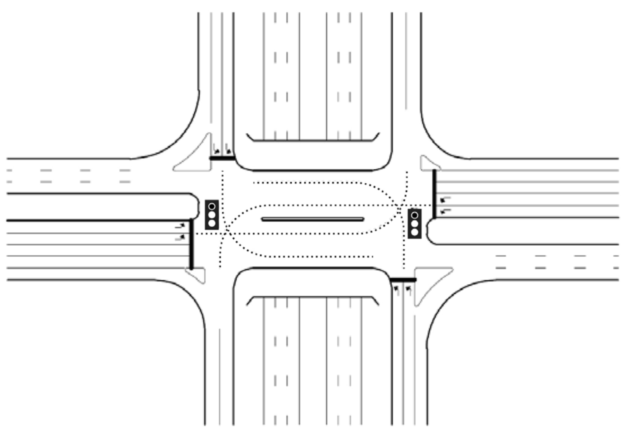

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Figure 26. Graphic. TDI.[9]

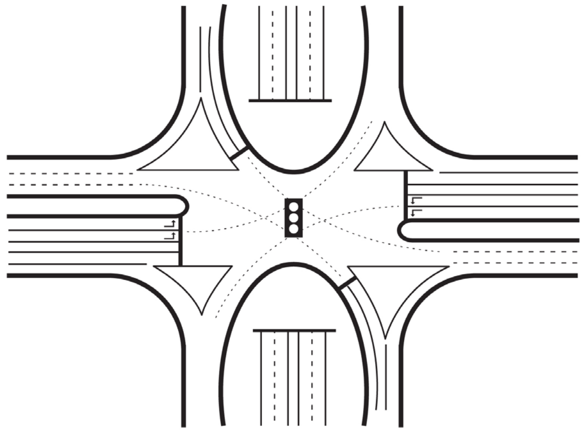

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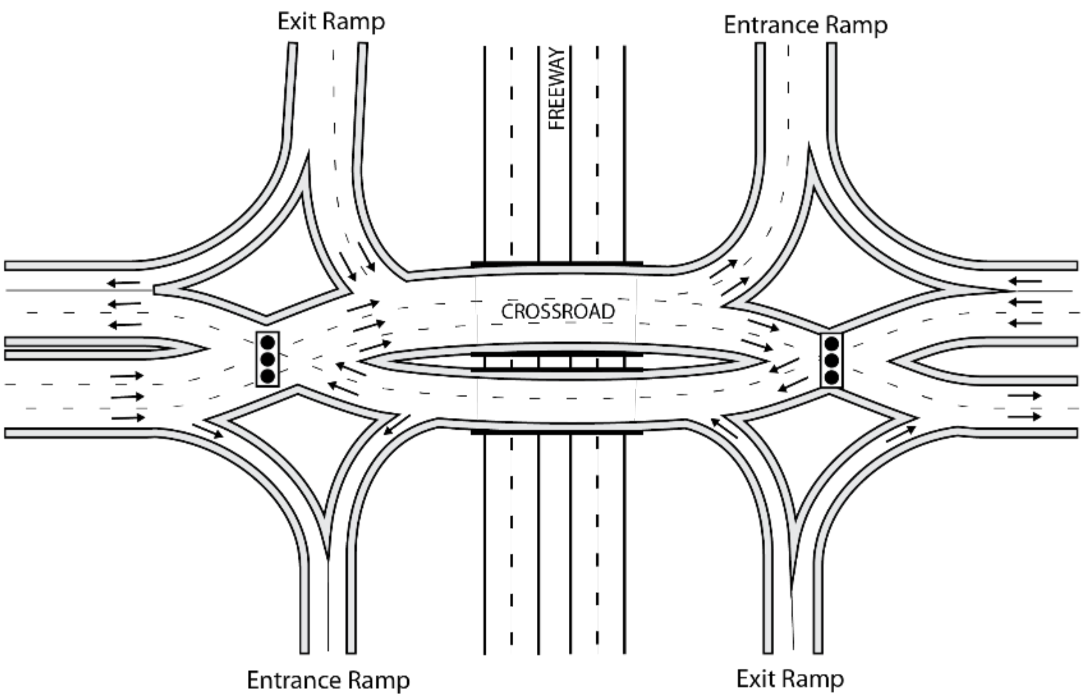
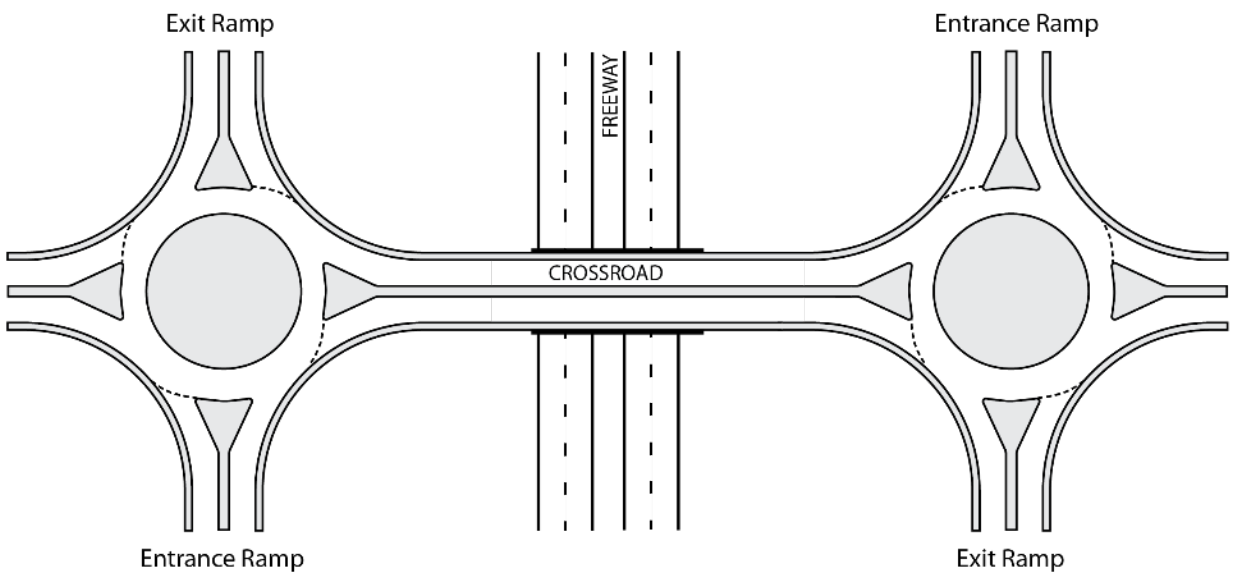
Figure 27. Graphic. SPDI.[(9)]



Source: FHWA.

Figure 28. Graphic. DDI ramp terminal.



Source: FHWA.

Figure 29. Graphic. Roundabout ramp terminal.

FREEWAY RAMP TERMINAL TYPE

This project considered two forms of merging and diverging movements: speed change lanes (acceleration and deceleration lanes) and lane adds/drops. Speed change lanes may be of the parallel or tapered form and are introduced near the interchange to facilitate the changes in speed required to navigate the ramp and the mainline. Both parallel and tapered speed change lanes are introduced or removed from the mainline using a taper. Lane adds and lane drops are auxiliary lanes that are added after the entrance of an on-ramp or removed from the freeway mainline at an exit ramp. In the cases of multilane ramps, there may be both a speed change lane and a lane add or lane drop. The project team further identified locations of any lane adds or lane drops within an interchange if not directly associated with an entrance or exit ramp. Lane adds and drops are provided at interchanges periodically and are often correlated with interchange type.

The project team also identified crossroad ramp terminals with merging or diverging movements.

FREEWAY RAMP TYPE

While there are many labels and definitions for ramps, the project team consolidated ramps into three standard forms for this project:

- Diamond ramp: A direct ramp between the crossroad ramp terminal to the mainline; it may include some curvature on the ramp.

- Loop ramp: A looping connection between a crossroad ramp terminal and the mainline; it includes a single or compound curve that results in a roughly 180-degree change in direction for the vehicle. The crossroad ramp terminal may be controlled or free flowing.

- Outer connection: This direct, free-flowing connection between the crossroad and the mainline is typically located on the outer portion of the interchange. The crossroad ramp terminal is usually free flowing.

Figure 30 provides an aerial image of an interchange that contains all three ramp types. The southwest and southeast quadrants of the interchange contain diamond ramps and loop ramps, while the northeast quadrant contains an outer connection ramp.

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Figure 30. Photo. Aerial view of an interchange containing diamond ramps, loop ramps, and direct connection ramps.[10]

DATA COLLECTION PRIORITIZATION

The project team collected data at existing interchanges of the types described in the Crossroad Ramp Terminal Types and Traffic Control, Freeway Ramp Terminal Type, and Freeway Ramp Type sections. The project prioritized States for inclusion in the dataset based on several factors, including the following:

- Geographic diversity: The project team identified States from across the United States to generalize the results.

- Large sample sizes: By choosing States with large sample sizes of the interchange types, the project team reduced the number of States needed for the project.

- Wide-scale implementation: The project team chose States with wide-scale implementation of any interchanges of the configuration described previously in figure 1.

- Available and integrable data: The project team chose States with easy-to-work-with roadway, traffic, and crash data.

- Accessible ramp traffic and crash data. Not every State has ramp data (particularly traffic data) available. While geometrics are less important for planning-level analysis, traffic data are the most important predictor of crash frequency and should be incorporated at all levels of interchange analysis.

Table 8 summarizes the States from which the project team collected data for each interchange configuration. The table provides the sample size collected within each State. The project team, to the extent possible, attempted to balance data collection across States and interchange types by State. However, for less common interchange configurations, such as DDIs, the project team collected data from all available locations that had at least 2 yr of data.

Table 8. Summary of number of responses by State for interchange data collection.

Interchange Configuration	Arizona	Missouri	North Carolina	Ohio	Utah	Total
Conventional diamond	11	11	9	13	19	63
CD	4	0	3	5	3	15
TDI	7	7	2	4	3	23
DDI	0	12	3	2	7	24
SPDI	13	9	7	3	16	48
Parclo A2	0	3	5	2	0	10
Parclo A4	0	1	7	7	1	16
Parclo B2	2	1	4	3	1	11
Parclo B4	0	0	6	2	0	8
Parclo AB2	4	9	11	9	2	35
Parclo AB4	2	0	2	5	1	10

DATA COLLECTION PROCESS

The data collection process consisted of the following:

- Identify interchange and document the location of key interchange components. The project team identified the locations of the following components at each interchange:

    - Freeway.
    - Crossroad.
    - Ramp segment.
    - Crossroad and freeway ramp terminals.

- Download the linear referencing system (LRS) for each State.
- Collect and integrate data from available roadway inventory databases, including key cross-section components and traffic volume data.
- Validate roadway inventory data and collect supplemental data using online satellite and street-level imagery to assist in data collection. Additionally, use geographic information system (GIS) software for supplemental data collection.
- Download crash files for all States from the Highway Safety Information System (HSIS) or State database, DOT GIS maps, and directly from the State's DOT office.(14)
- Integrate crash data using the LRS for interchange areas.

The following subsections provide further details for each step in the process, discussion of any challenges observed, and steps taken to overcome those challenges.

Identify Interchange Locations and Type

The project team first conducted a demonstration using North Carolina's data to identify the location and type of interchange and establish an approach to building an interchange area in GIS. The demonstration aimed to determine how much effort was required to find interchanges other than diamond and to develop a proof-of-concept for automating the process of building the interchange area.

The purpose of building an interchange area in GIS was to flag crashes mapped to GIS occurring within the interchange area since interchanges consist of multiple facilities. For example, a standard diamond interchange consists of the freeway section, the crossroad section, and four individual ramps. These sections would be identified as six distinct facilities within a roadway inventory with six distinct beginning and ending points for pulling crash statistics. On a large scale, identifying and pulling all crashes for an interchange would require a great deal of effort. Using an automated approach, the project team could spend more time identifying interchanges and capturing supplemental data.

Additionally, the North Carolina DOT provided a web-based application with interchanges mapped and classified by type and construction year. The project team exported these data to identify interchanges meeting certain criteria (e.g., opened to traffic for at least two of the study years). Additionally, the project team filtered the pool of interchanges to only those included in the study design to identify potential candidates for data collection. The demonstration effort showed that staff could efficiently identify and determine interchange types and that the GIS-based approach would work well for merging data, including crash data. The demonstration effort showed that annual crash frequency, for interchanges as a whole, was relatively consistent from year to year. Based on the results of the demonstration effort, the project team moved forward with data collection for all five States.

LRS Download

The data collection process included downloading the LRS for all five States in the study area to GIS software. The team could then visually connect the data in a GIS platform to a base map that served as a merging point for crash data. The project team identified interchange areas with the following criteria:

- Located 1,500 ft from the furthest painted gore on each side of the crossroad.
- Located 100 ft from the ramp curb return on the outside of each crossroad ramp terminal.

The project team used GIS to manually drop points and identify the interchange. Once each side of the freeway, crossroad, and ramps was identified, the team took the following steps to create a model to draw lines to connect the points:

1. Locate features along routes:
   a. Input route features: State LRS.
   b. Input features: Interchange points.
   c. Search radius: 10 ft.
   d. Route identifier field: RID.
   e. Event type: Point.

2. Add fields:
   a. Field name: From_Milepost.
   b. Field type: Double.
   c. Field name: To_Milepost.
   d. Field type: Double.

3. Calculate field:
   a. Field name: From_Milepost.
   b. Expression: From_Milepost = !MEAS!.

4. Calculate field:
   a. Field name: To_Milepost.
   b. Expression: To_Milepost = !MEAS!.

5. Make route event layer:
   a. Input route features: State LRS.
   b. Route identifier field: Unique route identity (ID) for the routes in the State.
   c. Input event table: Table output from step 4 of the model.
   d. Route identifier field: RID.
   e. Event type: Line.
   f. From_Measure field: From_Milepost.
   g. To_Measure field: To_Milepost.

6. Copy features: Copy the table output from step 5.

7. Dissolve:
    a. Dissolve fields: RID, interchange ID.
    b. Statistics fields: From_Milepost: Minimum.
    c. To_Milepost: Maximum.

8. Make route event layer:
    a. Input route features: State LRS.
    b. Route identifier field: Unique route ID for the routes in the State.
    c. Input event table: Table output from step 7 of the model.
    d. Route identifier field: RID.
    e. Event type: Line.
    f. From_Measure field: MIN_From_Milepost.
    g. To_Measure Field: MIN_From_Milepost.

9. Copy features: Copy the output from step 8. This output is a line file that displays the interchange area of study.

The project team identified several issues while running this model. The primary issue was that not all lines were drawn when running the model, which resulted from one of the following reasons:

- The LRS centerline ended between the points, or multiple LRS centerlines were in the interchange area. To overcome this problem, the project team added multiple points and reran the model to draw the lines or trace over the LRS and extend the line to the end of the study area.

- The LRS had multiple lines drawn on top of each other:

    - In Missouri and Arizona, the State LRS file had multiple lines drawn on top of each other for co-routes. For example, U.S. 40 and I–70 were drawn on top of each other.

    - To solve this issue, the project team highlighted the route, exported the route to a new layer, deleted one of the routes so that the model was identified, and drew a line for a single route only.

- For routes that contained a roundabout, the model had to rerun multiple times to draw the roundabout, or parts of the roundabout had to be manually traced and drawn.

Figure 31 shows an example interchange area mapped in GIS software after running the LRS-based model. While the freeway/ramp gores are not aligned on each side of the crossroad, the interchange area includes the same length in both directions of the freeway. The interchange area includes the entire freeway section within the identified bounds, all four ramps, and the entire length of the crossroad within the identified bounds.

Original map: © 2022 ESRI™, NC CGIA, Maxar. Modifications by FHWA (see Acknowledgments section).

Figure 31. Photo. Example interchange area in GIS.(15)

Once all the interchange areas were drawn, the project team identified overlaps between adjacent interchange areas. In these cases, the project team identified the halfway point between the overlap and moved the interchange lines to avoid overlap. Table 9 indicates the interchanges with overlap and the extent of the overlap.

Table 9. Interchanges that overlapped.

State	Interchange ID	Overlap (ft)	Halfway (ft)
Arizona	AZ 2 and AZ 23	605.36	302.68
	AZ 11 and AZ 12	150.48	75.24
	AZ 12 and AZ 13	302.57	151.285
	AZ 32 and AZ 33	782.85	391.425
	AZ 35 and AZ 36	2,097.17	1,048.585
	AZ 36 and AZ 40	2,354.78	1,177.39
	AZ 40 and AZ 37	2,092.21	1,046.105
Utah	UT 123 and UT 124	1,320.5	660.25
	UT 130 and UT 306	745.61	372.805
Ohio	OH 40 and OH 41	1,328.52	664.26
Missouri	MO 319 and MO 320	1,190.40	595.20
	MO 863 and MO 864	349.1	174.55

Collect and Integrate Roadway Inventory Data

The project team began the data collection and integration process by mapping existing roadway inventory data with freeways, segments, and ramps when available. General data elements collected from roadway inventory included functional class, number of through lanes, median type, AADT, and speed limit. The project team integrated LRS data based on segment IDs and mileposts, collected data for each facility component, and consolidated data as needed. For example, the project team collected each ramp's AADT volume and then aggregated the values into entrance ramp AADT, exit ramp AADT, and total AADT.

The project team used FHWA’s HSIS to download roadway and traffic data for North Carolina and Ohio facilities. The project team worked with Arizona, Missouri, and Utah DOTs to collect roadway and traffic data for facilities in these States.

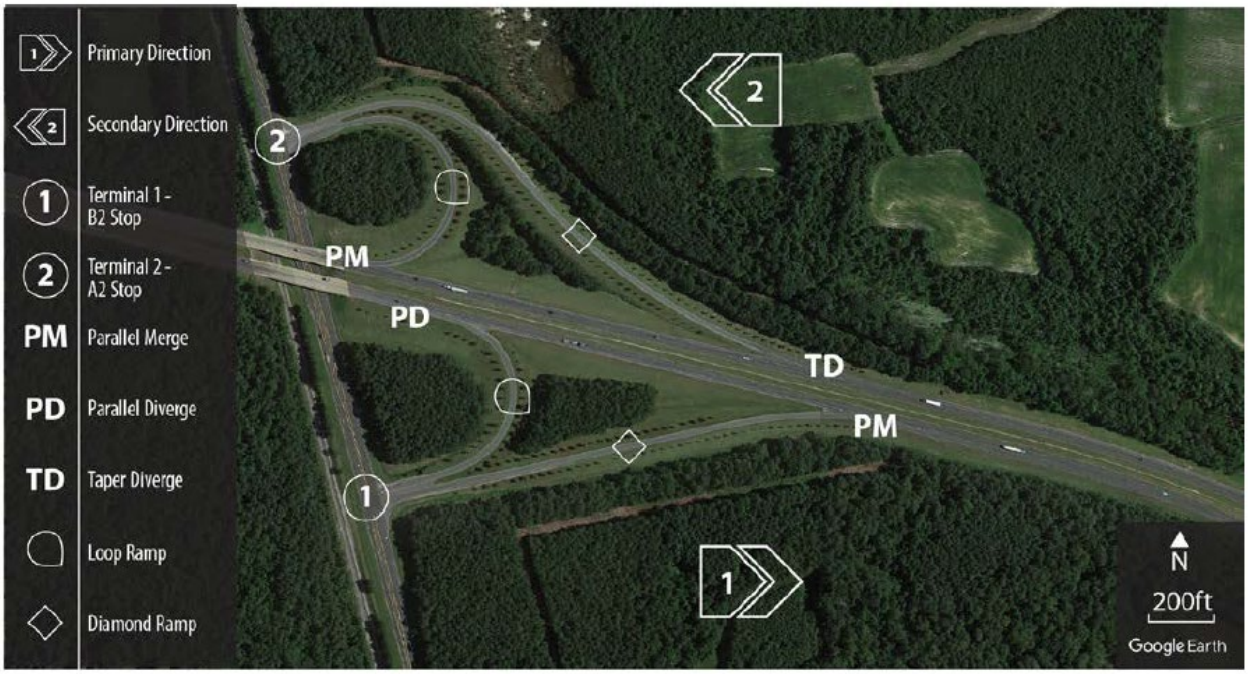
Roadway Inventory Validation and Supplemental Data Collection

The project team used satellite and street-level imagery, along with State video logs, to validate roadway inventory data and collect supplemental data outlined in the project’s data collection plan. Street-level imagery and video log files allowed the project team to verify the presence and amount of features (e.g., number of lanes on the major road), while Google Earth allowed the project team to verify the presence of features and to conduct measurements when necessary. Additionally, the project team took measurements in GIS software to support data collection as needed.

Figure 32 provides an example of the data collection process, including identification of the primary and secondary directions. This process facilitated data collection in both travel directions separately. The project team postprocessed directional data to determine the values of data elements in the crash frequency model interchange database.

In the United States, mileposts traditionally increase in the eastern direction for east-west routes and in the northern direction for north-south routes. In figure 32, this mainline is primarily an east-west route; therefore, the eastern direction is identified as the primary direction and the western direction as the secondary direction. For each interchange, the project team used route numbering to identify the primary direction (even-numbered routes are primarily east-west, odd-numbered routes are north-south) and guide the data collection.

Figure 32 provides an example where the entrance ramps have a parallel merge (PM), and the exit ramps have a parallel diverge and taper diverge. These features were individually coded into the data collection dataset and then processed in the analytical database to read as PMs and variable diverges. Additionally, figure 32 provides an overview of data collection by component, noting the presence of loop ramps and diamond ramps.



Original map: © 2021 Google® Earth™. Modifications by the authors (see Acknowledgments section).

Figure 32. Graphic. Sample component breakdown of a parclo AB2 interchange.[(10)]

Table 10 through table 12 provide an overview of the following:

- Table 10: Data collection elements collected for ramp segments.
- Table 11: Freeway and crossroad segments.
- Table 12: Crossroad ramp terminals.

Ramp data, in particular, were collected at the individual ramp level and aggregated into combined measures in the analysis dataset. As noted previously within the Collect and Integrate Roadway Inventory Data section, the project team looked at total entrance, total exit, and total ramp AADTs rather than individual ramp AADTs. The project team also computed the coefficient of variation (COV) for ramp AADTs to look at the impact of flow differential among ramps and its impact on safety performance.

Table 10. Data elements for ramp segments.

Category	Attribute	Description
General	State	Indicator for State
	Interchange ID	Unique interchange identifier
	GPS_loc	GPS latitude and longitude
	Ramp_type	Indicator for entrance or exit ramp
Satellite imagery street-level imagery GIS software	Ramp_length	Ramp length in miles
	Ramp_fr_type	Ramp freeway terminal type (parallel or taper)
	Ramp_side	Indicator for ramp side (left or right)
	Ramp_meter	Indicator for presence of ramp meter
	Ramp_lanes	Number of ramp through lanes
State agency LRS	Ramp_AADT	Ramp AADT

GPS = Global Positioning System.

Table 11. Data elements for freeway and crossroad segments.

Category	Attribute	Description
General	State	Indicator for State
	Interchange_ID	Unique interchange identifier
	Freeway_ID	Freeway route number
	Crossroad_ID	Crossroad name or route number
	GPS_loc	GPS latitude and longitude
	Config	Interchange configuration
	Area_type	Urban or rural area type
Satellite imagery street-level imagery GIS software	Int_skew	Skew angle between freeway and crossroad
	Xr_loc	Overpass or underpass crossroad location
	Ml	Indicator for presence of a managed lane
	Fr_len	Freeway length in interchange area
	Quadrants	Interchange number of quadrants (two or four)
	Parclo_T	Parclo type (A, B, or AB)
	Xr_len	Crossroad length in interchange area
	Term_len	Length between crossroad terminal midpoints
	Cd_rd	Indicator for presence of a collector-distributor road
	Min_adj_int_Xr	Minimum adjacent crossroad-to-crossroad distance
	Avg_adj_int_Xr	Average adjacent crossroad-to-crossroad distance
	Min_adj_int_gore	Minimum adjacent gore-to-gore distance
	Avg_adj_int_gore	Average adjacent gore-to-gore distance
	Min_adj_sig_int_	Minimum adjacent terminal to signal distance
	Avg_adj_sig_int	Average adjacent terminal to signal distance
	Weave_ind	Indicator for weaving section at interchange boundary
	Ln_add_drop	Indicator for lane add or drop presence
	Adj_land_use	Primary adjacent land use
	Xr_sidewalk	Indicator for presence of sidewalk on crossroad
	Xr_bike	Indicator for presence of bicycle facility on crossroad
	Light	Indicator for presence of interchange lighting
	Cons_Yr	Indicator for construction by year at interchange
State agency LRS	Free_fun_class	Freeway functional classification
	Xr_fun_class	Crossroad functional classification
	Fr_lanes	Number of freeway through lanes
	Xr_lanes	Number of crossroad through lanes
	Fr_med_type	Freeway median type
	Fr_AADT	Bidirectional freeway AADT
	Xr_AADT	Bidirectional crossroad AADT
	Fr_PSL	Freeway posted speed limit
	Xr_PSL	Crossroad posted speed limit

Table 12. Data elements for crossroad ramp terminals.

Category	Attribute	Description
General	State	Indicator for State
	Interchange ID	Unique interchange identifier
	GPS_loc	GPS latitude and longitude
	Xr_term_ID	Unique crossroad ramp terminal identifier
	Xr_term_type	Crossroad ramp terminal type
Satellite imagery street-level imagery	Xr_term_ctrl	Crossroad ramp terminal traffic control type (signal, all-way stop, ramp stop, yield, roundabout, no control)
	Xr_term_lt_sig	Crossroad ramp terminal left-turn signalization type (protected, permitted, protected/permitted, N/A)
	Xr_term_rt_ctrl	Crossroad ramp terminal right-turn control (signal, stop, yield, no control)
	Xr_term_rt_chan	Indicator for crossroad ramp terminal right-turn channelization
	Xr_term_lt_lane	Crossroad ramp terminal crossroad left-turn lanes
	Xr_term_rt_lane	Crossroad ramp terminal crossroad right-turn lanes
	Xr_term_ramp_lt	Crossroad ramp terminal ramp left-turn lanes
	Xr_term_ramp_rt	Crossroad ramp terminal ramp right-turn lanes
	Xr_term_nonramp	Indicator for nonramp crossroad ramp terminal leg
	Xr_term_dway	Number of driveways within 250 ft of crossroad ramp terminal on outside legs
	Xr_term_acc_ln	Presence of acceleration lane at crossroad ramp terminal
	Xr_term_decel_ln	Presence of deceleration lane at crossroad ramp terminal
	Xr_term_cr_wlk	Presence of painted crosswalk at crossroad ramp terminal
	Xr_term_ped_ln	Number of lanes crossed by pedestrians on crossroad
	Xr_ramp_ped_ln	Number of lanes crossed by pedestrians on ramps
	Xr_term_ped_sig	Presence of pedestrian signals at crossroad ramp terminals
	Xr_term_ped_rt	Number of uncontrolled right-turns conflicting with pedestrian crossings
	Xr_term_frontage	Indicator for frontage road
State agency LRS	Xr_term_nr_aadt	Crossroad ramp terminal nonramp leg AADT

N/A = not applicable.

Crash Data Download

As with the roadway inventory and traffic data, the project team used FHWA's HSIS to download crash data for North Carolina and Ohio facilities.(14) Arizona, Missouri, and Utah DOTs provided crash data for facilities in their States. The project team collected 2014 through 2020 crash data for Arizona; 2016 through 2020 data for Missouri, North Carolina, and Ohio; and

2013 through 2020 for Utah. The project team integrated and used the most recent 5-yr (2016 through 2020) data for each State.

The project team used satellite and street-level historic imagery at each site to confirm the interchange existed in its present form for the entire duration of the study period.[10] If an interchange had undergone construction during the study period, the years of construction activity were dropped from the dataset. The project team removed any interchanges from the dataset if at least 2 full yr of data were not available. Additionally, for newer interchange configurations (such as the DDI), the interchange was included if it was open for at least 2 yr during the study period.

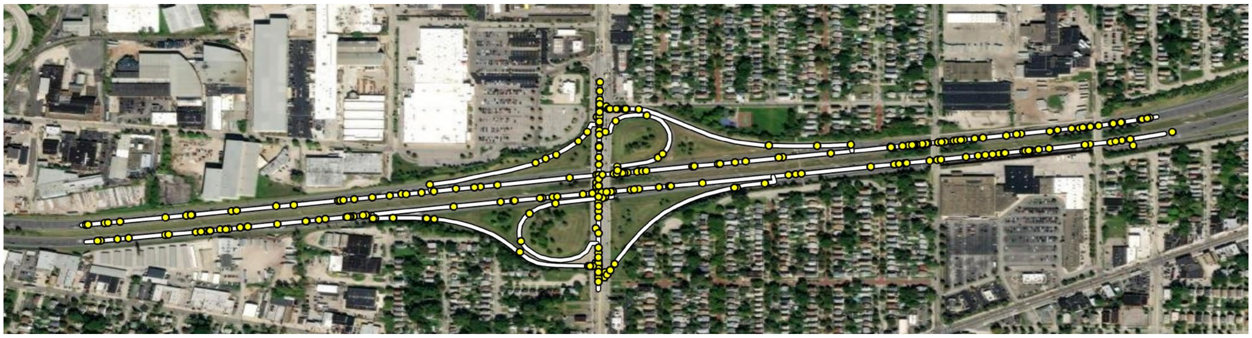
The project team grouped crashes into KABC crashes and PDO crashes for crash frequency and severity model development. The project team classified crashes based on the KABCO scale, defining K, A, B, or C as "fatal and injury" and O as "PDO" crashes.

Crash Data Integration

Once all the interchange lines were drawn, the project team integrated crash data. Crash data came as spreadsheet data files and GIS point files. The following were the steps taken for each of the crash files:

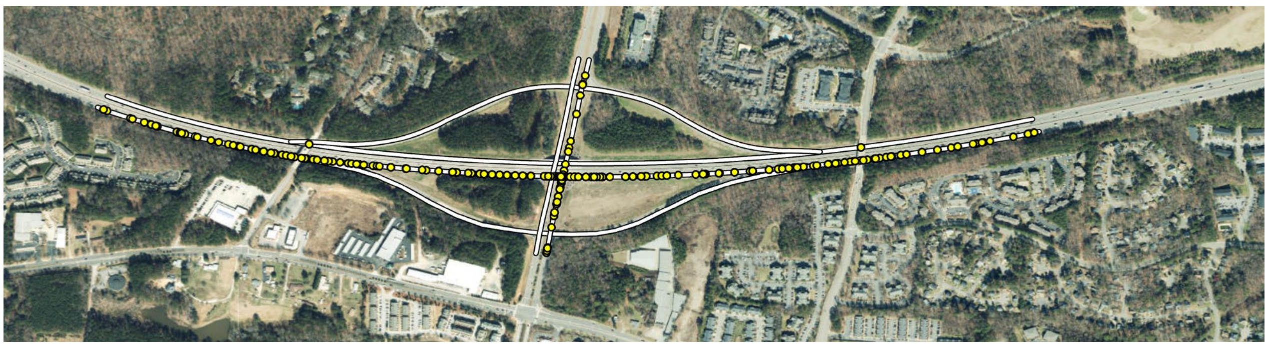
- GIS point files—Perform a spatial join:
  - Target features: Crash file.
  - Join features: Interchange line file.
  - Output feature class: State_Crash.
  - Join operation: Join one-to-one.
  - Uncheck the box "Keep All Target Features."
  - Match option: Intersect.
  - Search radius: 20 ft.
- Excel files:
  - Make route event layer:
    - Input route features: State LRS.
    - Route identifier field: Unique State route identifier.
    - Input table: Crash file.
    - Route identifier field: Unique State route identifier.
    - Event type: Point.
    - Measure field: Milepost field in crash file.
    - Layer name: State crashes.
  - Perform a spatial join.

Figure 33 provides a sample interchange with crash points merged to the interchange area. Arizona, Ohio, and Utah all had crashes reported on both inventory and noninventory directions of the road and onramps. Missouri's crashes were all associated with the interchange's latitude and longitude. North Carolina only reported crashes on the inventory side of the roadway. Crashes were not represented on ramps or the noninventory side. Figure 34 provides an example graphic of an interchange in North Carolina where crashes are only represented on the inventory side of the roadway. This situation did not cause data collection issues since the database treated each interchange as the unit of observation rather than the component of the interchange.



Original map: © 2022 ESRI™, NC CGIA, Maxar. Modifications by FHWA (see Acknowledgments section).

Figure 33. Photo. Interchange with crash points.(15)



Original map: © 2022 ESRI™, NC CGIA, Maxar. Modifications by FHWA (see Acknowledgments section).

Figure 34. Photo. Interchange showing crashes on the inventory side of the roadway.(15)

The project team encountered some issues merging the GIS crash files. For example, some States had crashes that were located more than 20 ft away from the roadway's centerline. The project team had to add these crashes into the crash file. In North Carolina, two interchanges had crashes reported on both the inventory and noninventory side of the roadway. The interchanges that had noninventory crashes were NC 273 and NC 948. In addition, NC interchange 560 had one crash reported on a ramp.

DATABASE SUMMARY

This section summarizes the data assembled for estimating planning-level predictive models for interchanges by type. As noted, the project team originally anticipated developing separate databases for each interchange configuration, but this would have led to sample size issues and possible counterintuitive results for safety performance effects for features common to multiple interchange configurations. The crash frequency database consists of one master database file with separate outcome variables for KABC crashes and PDO crashes.

The project team separately arranged the data into an SDF development dataset. This dataset treats the crash as the outcome of interest and interchange features, including configuration, as input variables for the CPM. This dataset used the same data as the crash frequency database, and the resulting models can be used together to predict crash frequency at the interchange level for specific crash severity.

Table 13 provides summary data, including AADT range and number of KABC and PDO crashes by interchange configuration. Not all interchange configurations had sufficient sample sizes to estimate separate predictive models. Therefore, a pooled approach was taken, using a minimum of 300 crashes per year. This approach is consistent with the recommendation by Srinivasan and Bauer to include at least 300 crashes per year for the crash type and severity category of interest.(16)

Table 13. Summary data for geometric, operational, and crash data.

Int. Config.	Freeway AADT Range	Crossroad AADT Range	Entrance Ramp AADT	Exit Ramp AADT	KABC Crashes	PDO Crashes
Diamond	5,000–210,000	350–40,500	100–33,400	125–24,500	2,139	7,157
CD	23,100–236,000	11,000–52,900	6,800–25,500	4,250–22,500	1,246	4,278
TDI	17,000–207,300	3,200–55,000	4,000–36,500	4,500–36,700	4,552	13,659
DDI	29,000–191,000	2,000–47,000	2,000–38,500	2,000–45,000	4,037	10,883
SPDI	21,000–261,000	3,700–64,000	3,100–70,000	3,200–75,000	11,964	38,102
Parclo A2	6,400–115,300	1,500–30,615	650–9,400	1,300–21,800	673	1,283
Parclo A4	46,181–135,000	12,000–68,000	10,200–34,400	9,300–39,600	1,032	3,663
Parclo B2	7,298–123,000	150–32,000	35–14,800	35–12,400	744	1,789
Parclo B4	23,900–144,000	1,200–67,500	4,900–32,200	4,300–31,000	555	1,855
Parclo AB2	5,500–300,000	200–51,500	200–29,200	200–24,600	3,144	9,841
Parclo AB4	22,000–132,300	9,000–57,000	5,600–27,600	5,500–27,200	423	1,253

Int. Config. = interchange configuration.

Table 14 provides an overview of the summary statistics for continuous variables for geometric, operational, and crash data. The summary statistics include the average (or mean), standard deviation, minimum, and maximum values for each characteristic. The project team used summary statistics for the initial screening of elements for potential outliers in the dataset. The project team further scrutinized or removed sites with unknown or extreme values from the dataset.

Table 14. Summary information for geometric, operational, and crash data.

Variable	Average	St. Dev.	Minimum	Maximum
Interchange skew angle (degrees)	12.84	14.13	0	60
Freeway through lanes (both directions) (No.)	5.84	2.11	4	12
Crossroad through lanes (both directions) (No.)	3.77	1.30	2	6
Freeway AADT (No.)	79,612	59,593	5,028	300,000
Crossroad AADT (No.)	20,500	15,617	168	68,000
Freeway posted speed limit (mph)	65.19	5.63	35	75
Crossroad posted speed limit (mph)	42.02	9.02	20	65
Interchange length along freeway (miles)	0.57	0.19	0.1	1.15
Interchange length along crossroad (miles)	0.21	0.09	0.07	0.59
Crossroad ramp terminal separation (miles)	0.15	0.09	0	0.8
Adjacent crossroad minimum distance (miles)	1.89	1.44	0.36	9.65
Adjacent crossroad average distance (miles)	2.52	1.78	0.51	10.45
Adjacent gore minimum distance (miles)	1.38	1.41	0.13	8.97
Adjacent gore average distance (miles)	2.00	1.76	0.17	9.74
Adjacent intersection minimum distance (miles)	3.83	37.98	0.05	550
Adjacent intersection average distance (miles)	5.82	47.53	0.05	710
Combined exit ramp length (miles)	0.61	0.21	0.19	1.46
Combined entrance ramp length (miles)	0.64	0.25	0.20	1.91
Combined entrance ramp AADT	13,400	11,229	33	69,985
Combined exit ramp AADT	13,383	11,210	15	74,985
Driveway access points (No.)	0.69	1.27	0	7
Maximum number of lanes crossed by pedestrians	4.82	2.48	0	10
Pedestrian right-turn conflicts (No.)	0.48	1.06	0	7
Interchange area (mi^2)	0.12	0.07	0.01	0.42
Fatal and injury crashes per year	29.00	44.23	0	228.60
PDO crashes per year	89.13	140.66	0	746.80

St. Dev. = standard deviation; No. = Number of.

Table 15 provides an overview of the summary data for frequency variables included in the final dataset. The project team used these variables primarily as indicators in the dataset to describe the presence or absence of features or to distinguish between geometric design characteristics. The project team used indicator frequencies to the extent possible to better understand the sample size underlying the coefficients estimated in the model to attempt not to include factors with very small samples. In some cases, continuous data provided in table 14 were converted to categorical variables when indicated by preliminary data analysis. For example, the project team identified that a skew angle of 30 degrees or greater was associated with a difference in safety performance relative to interchanges with skew less than 30 degrees.

Table 15. Summary information for categorical data.

Category	Characteristic	Percent Frequency
Ramp meter	Present	13.69
	Not present	86.31
Nonramp leg at terminal	Present	7.22
	Not present	92.78
Crossroad ramp terminal control	All-way stop	0.40
	Ramp stop	22.81
	Roundabout	6.84
	Signal	69.58
	Yield	0.38
Area type	Rural	23.19
	Urban	76.81
Crossroad location	Overpass	56.65
	Underpass	43.35
Weaving section at interchange area boundary	Present	14.45
	Not present	85.55
Frontage road	Present	9.20
	Not present	90.80
Sidewalk	Present	45.63
	Not present	54.37
Lighting	Present	80.61
	Not present	19.39
Freeway median type	Rigid barrier	41.06
	Semirigid barrier	23.57
	Flexible barrier	12.17
	Depressed/grass	23.20
Collector distributor road	Not present	96.96
	Present	3.04
Managed lane	Not present	85.17
	Present	14.83

CHAPTER 5. PREDICTIVE MODEL DEVELOPMENT

INTRODUCTION

This chapter describes the development of the planning-level CPMs and the overall findings for interchange safety performance by configuration. It includes an overview of the methods considered and employed for developing statistical models, the final variables included in the CPMs (and their interpretations), and the final models recommended for assessing the safety performance of interchange configurations and associated severity distributions.

The CPMs in this chapter may be used to predict crash frequency for an interchange area as defined in chapter 3. The CPM predicts all crashes within the interchange area and does not provide a breakdown by facility component (i.e., ramp terminals, ramps, freeway segment, or crossroad segment). The CPM includes an SPF, a set of AFs, a calibration factor, and an SDF. The project team designed this methodology to be consistent with the project design-level approach provided in Part C of the HSM.[3] While the HSM Part C focuses on individual design-level components, this application focuses on planning-level inputs and interchange area crash frequency and severity analysis.

The interchange area may not be the same size for all alternatives considered. For a fair comparison, alternative analysis should consider the largest interchange area or focus on the interchange influence area as the baseline for all analyses. The HSM Part C predictive method can supplement this analysis for other components of the interchange influence area or for the difference in interchange area along the freeway mainline or crossroad.[3]

MODELING APPROACH

The project team worked with FHWA to decide on a model form to fit within the project constraints and yield the most reliable predictions possible. Since the CPM includes a pooled set of interchanges across varying configurations, the model development databases include at least 300 crashes per year, which adheres to recommendations by Srinivasan and Bauer.[16] Additionally, because the project team was developing SDFs, there was less of a need to develop separate CPMs for each crash severity level.

The project team began by developing the KABC crash predictive model and then used those results to develop a PDO crash predictive model. Once the project team completed the KABC model, they tested the AFs in the PDO crashes model. The project team further assessed the merits of predictors not included in the KABC model, particularly those expected to influence PDO crash frequency.

As indicated in chapter 3, the project team used VIEDA, as outlined in *The Art of Regression Modeling in Road Safety*, to select and include variables in the predictive model.[11] The project team retained variables, or AFs, in the KABC and PDO models if the VIEDA identified a relationship with safety outcomes and if the magnitude of effect was consistent with intuition. The project team did not hold to a strict confidence level when assessing statistical significance for inclusion in the models.

Due to the overall sample size of data collected for CPM development, the project team could not use a hold-out process for model validation. Ideally, the project team would have withheld data from one State to evaluate model fit and transferability. However, since such a broad range of interchange configurations was gathered, the project team had too few sites to withhold for any one State. Further, using a 70-percent training and 30-percent testing dataset for calibration was considered but deemed impractical due to overall sample sizes. Therefore, the project team relied on model fit statistics and CURE plots to assess the validity of the models.

STATISTICAL ANALYSIS METHODS

The current state of the practice for developing statistical road safety models is to assume a log-linear relationship between expected crash frequency and site characteristics. Generalized linear modeling techniques were applied to develop the models, and a log-linear relationship was specified using a negative binomial error structure. The negative binomial error structure allows for overdispersion that is often present in crash data (i.e., the variance is greater than the mean). Typically, in statistical road safety modeling, this "additional dispersion" is specified as a dispersion parameter multiplied by the expected number of crashes squared (NB-2 model).

The models considered for this analysis were structured as shown in figure 35.

$$\lambda_i = \exp^{(\beta_0 + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_n X_n + \varepsilon_i)}$$

Figure 35. Equation. Structure of prediction models.

Where:

λ_i = expected number of crashes for the *i*th observation, with an observation in this analysis being a segment year.

X_i = a matrix of explanatory variables associated with λ_i, including traffic volume (AADT), geometric design features, and operational characteristics.

β = a vector of parameters to be estimated that quantify the relationships between the explanatory variables and λ_i.

ε_i = a disturbance term, where $\exp(\varepsilon_i)$ is gamma-distributed, with a mean equal to one and variance equal to α_i.

The NB-2 model's variance in the expected number of crashes is then written as $\lambda_i + \alpha_i\lambda_i^2$. The base SPFs and AFs are inferred from the resulting NB-2 model.

FATAL AND INJURY CRASH FREQUENCY PREDICTION MODEL

This section describes the development of predictive model equations, based on KABC crash data, in the following five subsections:

- The Model Development subsection describes the structure of the predictive equations as used in the regression analysis.

- The Model Estimation subsection describes the regression statistics for each of the estimated equations (i.e., the proposed safety prediction models).

- The Model Evaluation subsection briefly describes the model fit parameters used to assess model validity.

- The Estimated AFs subsection describes the estimated AFs.

- The Sensitivity Analysis subsection provides a sensitivity analysis of the predictive model equations over the applicable range of AADT.

Model Development

The regression model predicts KABC crash frequency for the interchange area. The generalized form of the CPM shows all the AFs in the model. Indicator variables determine which AFs are applicable to each observation in the dataset based on the associated interchange configuration type.

The generalized form consists of AFs with indicator or continuous variables used to determine when the AF is applicable or applicable to a specific extent. For example, the generalized form of the area type AF includes an indicator variable that is used to determine when the subject interchange is applicable. The number of left-turn lanes on crossroad approaches to crossroad ramp terminals is considered a base condition of zero-turn lanes.

Figure 36 through figure 46 describe the generalized regression model.

$$N_{p,kabc} = y \times N_{spf,kabc} \times AF_{fr56,kabc} \times AF_{fr>6,kabc} \times AF_{xr>4,kabc} \times AF_{at,kabc} \times AF_{skew,kabc} \times AF_{gore,kabc} \times AF_{ml,kabc} \times AF_{xrlt,kabc} \times AF_{curvol,kabc}$$

Figure 36. Equation. Generalized KABC CPM.

with

$$N_{spf,kabc} = \exp\left(b_{0,kabc} + b_{1,kabc} \times ln\left[\frac{AADT_{fr}}{In_{fr}} \times AADT_r\right] + b_{2,kabc} \times ln\left[\frac{AADT_{xr}}{In_{xr}}\right] + b_{3,kabc} \times I_{DDI} + b_{4,kabc} \times I_{Par\,clo} + b_{5,kabc} \times I_{Par\,clo\,A} + b_{6,kabc} \times I_{SPDI} + b_{7,kabc} \times I_{TDI} + b_{8,kabc} \times I_{RD} + b_{9,kabc} \times ln\left[\frac{AADT_{fr}}{In_{fr}} \times AADT_r\right] \times I_{SPDI} + b_{10,kabc} \times ln\left[\frac{AADT_{xr}}{In_{xr}}\right] \times I_{TDI} + b_{11,kabc} \times ln\left[\frac{AADT_{xr}}{In_{xr}}\right] \times I_{SPDI}\right)$$

Figure 37. Equation. KABC SPF.

$$AF_{fr56,kabc} = \exp\left(b_{fr56,kabc} \times I_{fr56}\right)$$

Figure 38. Equation. Freeway AF for five and six lanes.

$$AF_{fr>6,kabc} = \exp\left(b_{fr>6,kabc} \times I_{fr>6}\right)$$

Figure 39. Equation. Freeway AF for more than six lanes.

$$AF_{xr>4,kabc} = \exp\left(b_{xr>4,kabc} \times I_{xr>4}\right)$$

Figure 40. Equation. Crossroad AF for more than four lanes.

$$AF_{at,kabc} = \exp\left(b_{at,kabc} \times I_{at}\right)$$

Figure 41. Equation. Area type AF.

$$AF_{skew,kabc} = \exp\left(b_{skew,kabc} \times I_{skew}\right)$$

Figure 42. Equation. Interchange skew AF.

$$AF_{gore,kabc} = \exp\left(b_{gore,kabc} \times I_{gore}\right)$$

Figure 43. Equation. Nearby gore AF.

$$AF_{ml,kabc} = \exp\left(b_{ml,kabc} \times I_{ml}\right)$$

Figure 44. Equation. Managed-lane AF.

$$AF_{xrlt,kabc} = \exp\left(b_{xrlt,kabc} \times LT_{xr}\right)$$

Figure 45. Equation. Crossroad left-turn lanes AF.

$$AF_{cvrvol,kabc} = \exp\left(b_{cvrvol,kabc} \times CV_{rvol}\right)$$

Figure 46. Equation. Ramp volume COV AF.

Where:

- $AADTfr$ = freeway bidirectional AADT volume (vehicles/day).
- $AADTr$ = total ramp AADT volume (vehicles/day).
- $AADTxr$ = crossroad bidirectional AADT volume (vehicles/day).
- $AFat,kabc$ = AF for area type for KABC crashes.
- $AF_{cvrvol,kabc}$ = AF for COV for ramp volumes for KABC crashes.
- $AF_{fr56,kabc}$ = AF for five or six freeway through lanes for KABC crashes.
- $AF_{fr>6,kabc}$ = AF for more than six freeway through lanes for KABC crashes.
- $AF_{gore,kabc}$ = AF for close gore-to-gore interchange spacing for KABC crashes.
- $AF_{ml,kabc}$ = AF for managed lane presence for KABC crashes.
- $AF_{skew,kabc}$ = AF for freeway and crossroad skew for KABC crashes.
- $AF_{xr>4,kabc}$ = AF for more than four crossroad through lanes for KABC crashes.
- $AF_{xrlt,kabc}$ = AF for the number of crossroad left-turn lanes for KABC crashes.
- b_i = regression coefficient for condition i.
- CV_{rvol} = COV for ramp volumes.

I_{at} = indicator variable for area type (1 = urban; 0 otherwise).
I_{DDI} = indicator variable for DDI interchange configuration (1 = DDI; 0 otherwise).
I_{fr56} = indicator variable for five or six freeway through lanes (1 = yes; 0 otherwise).
$I_{fr>6}$ = indicator variable for more than six freeway through lanes (1 = yes; 0 otherwise).
I_{gore} = indicator variable for gore-to-gore spacing with nearest adjacent interchange less than 0.5 mi (1 = yes; 0 otherwise).
I_{ml} = indicator variable for the presence of managed lane(s) in one or both directions (1 = yes; 0 otherwise).
I_{Parclo} = indicator variable parclo interchange configuration (1 = parclo; 0 otherwise).
$I_{ParcloA}$ = indicator variable parclo type A interchange configuration (1 = parclo A; 0 otherwise).
I_{RD} = indicator variable for roundabout diamond interchange configuration (1 = roundabout diamond; 0 otherwise).
I_{skew} = indicator variable for skew angle between freeway and crossroad greater than 30 degrees (1 = yes; 0 otherwise).
I_{SPDI} = indicator variable for SPDI interchange configuration (1 = SPDI; 0 otherwise).
I_{TDI} = indicator variable for TDI interchange configuration (1 = TDI; 0 otherwise).
$I_{xr>4}$ = indicator variable for more than four crossroad through lanes (1 = yes; 0 otherwise).
In_{fr} = number of freeway through lanes.
In_{xr} = number of crossroad through lanes.
LT_{xr} = number of left-turn lanes on crossroad approaches.
$N_{p,kabc}$ = predicted average KABC crash frequency (crashes/year).
$N_{spf,kabc}$ = predicted average KABC crash frequency for an interchange with base conditions (crashes/year).
y = study period (years).

The regression equations include the base condition for the attribute. The base conditions were established as those most commonly occurring in the study sites and included the following:

- Interchange configuration: Conventional diamond or CD interchange.
- Number of freeway through lanes: Four.
- Number of crossroad through lanes: Two, three, or four.
- Area type: Rural.
- Interchange skew: <30 degrees.
- Minimum gore-to-gore distance to nearby interchange: ≥0.50 mi.
- Managed lanes in one or both directions: Not present.
- Ramp AADT COV: 0.
- Crossroad number of turn lanes at crossroad ramp terminals: 0.

Model Estimation

As indicated in the Model Development section, the project database included all interchange configurations in a pooled dataset. With this approach, the regression model was used to develop figure 36 through figure 46 from a single equation. The coefficients for variables specific to each interchange configuration are pulled from the model. Variables not specific to an interchange configuration are estimated with data from all configurations. This approach constrains the impacts of these variables to be the same across interchange configurations.

Table 16 provides the generalized negative binomial regression model results. The final model includes 261 interchanges in the pooled dataset. The model fit statistics indicate an improvement in the overall fit by including the associated predictors.

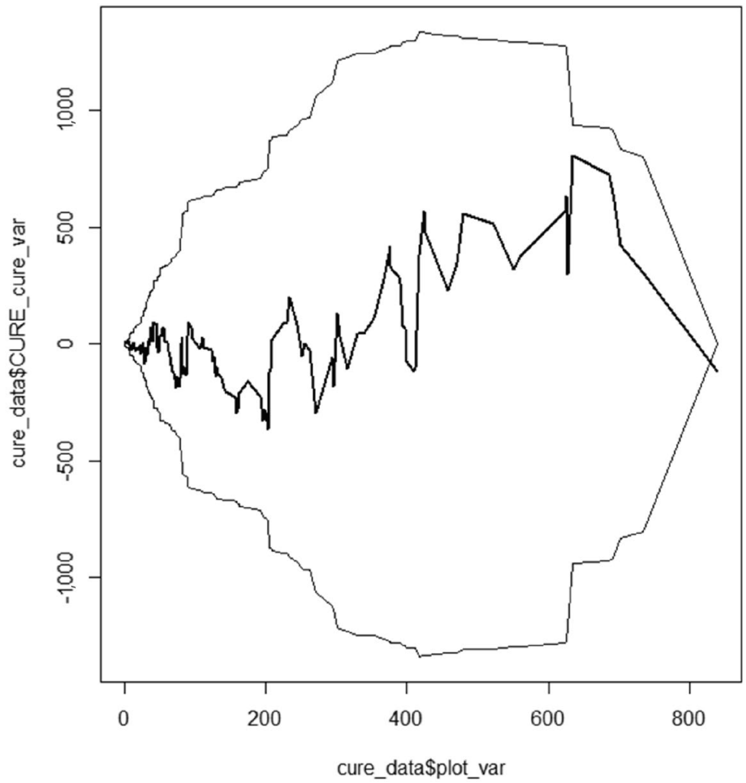
Table 16. Final predictive model and parameters for KABC crash frequency.

Variable	Description	Value	Standard Error	*z*-statistic
$b_{0,kabc}$	Interchange constant	−6.814	0.658	−10.35*
$b_{1,kabc}$	Freeway volume per lane interaction with ramp volume	0.376	0.046	8.15*
$b_{2,kabc}$	Crossroad volume per lane	0.189	0.072	2.63*
$b_{3,kabc}$	DDI main effect	−0.083	0.150	−0.55
$b_{4,kabc}$	Parclo interchange main effect	0.158	0.104	1.52
$b_{5,kabc}$	Parclo type A interchange additive effect	−0.221	0.132	−1.67
$b_{6,kabc}$	SPDI main effect	−5.563	2.088	−2.66*
$b_{7,kabc}$	TDI main effect	−3.064	1.573	−1.95*
$b_{8,kabc}$	Roundabout diamond interchange main effect	−0.267	0.164	−1.63
$b_{9,kabc}$	Interaction between SPDI and freeway volume per lane	0.214	0.122	1.76
$b_{10,kabc}$	Interaction between TDI and crossroad volume per lane	0.362	0.183	1.97*
$b_{11,kabc}$	Interaction between SPDI and crossroad volume per lane	0.151	0.166	0.91
$b_{fr56,kabc}$	Freeway five or six through lanes	0.363	0.101	3.59*
$b_{fr>6,kabc}$	Freeway with more than six through lanes	0.744	0.125	5.93*
$b_{xr>4,kabc}$	Crossroad more than four through lanes	0.227	0.091	2.49*
$\mathrm{b}_{at,kabc}$	Urban area type	0.367	0.118	3.12*
$b_{gore,kabc}$	Minimum nearby gore distance within 0.5 mi	0.206	0.128	1.61
$b_{ml,kabc}$	Managed lane presence	0.282	0.143	1.97*
$b_{xrlt,kabc}$	Crossroad left-turn lanes	−0.056	0.048	−1.19
$b_{cvrvol,kabc}$	Ramp volume variability	−0.299	0.148	−2.01*
$b_{skew,kabc}$	Interchange skew angle	0.235	0.100	2.34*

*Significant at 0.05 level.
Overdispersion parameter: 0.242.

Model Evaluation

The project team evaluated the validity of the models through CURE plots to maximize the sample size used for model development. Figure 47 provides the CURE plot for KABC crashes. As shown, the CUREs fall within the confidence intervals for the entire range of predicted crashes. The CUREs generally oscillate about zero, indicating minimum bias for any range of predicted crash frequency. Overall, the CURE plot indicates a good fit between the predicted and observed data.



Source: FHWA.

Figure 47. Graph. CURE plot for KABC crashes.

Estimated AFs

The project team estimated several AFs in conjunction with the SPFs using KABC crash data. Collectively, they describe the relationship between various factors and crash frequency. This section describes the AFs, including the range of applicability.

Freeway Through Lanes

Figure 48 and figure 49 provide the number of freeway through lanes AFs.

$$AF_{fr56,kabc} = \exp\,(0.363 \times I_{fr56})$$

Figure 48. Equation. Freeway with five and six lanes KABC AF value.

$$AF_{fr>6,kabc} = \exp\,(0.744 \times I_{fr>6})$$

Figure 49. Equation. Freeway with more than six lanes KABC AF value.

The base condition is four freeway through lanes. The AF shows an expected 43.8-percent increase in KABC crashes when five or six freeway through lanes are present. The AF shows an expected 110.4-percent increase in KABC crashes when more than six freeway through lanes are present.

Crossroad Through Lanes

Figure 50 provides the number of crossroad through lanes AF.

$$AF_{xr>4,kabc} = \exp\left(0.227 \times I_{xr>4}\right)$$

Figure 50. Equation. Crossroad through lanes KABC AF value.

The base condition is four or fewer crossroad through lanes. The AF shows an expected 25.5-percent increase in KABC crashes when five or more crossroad through lanes are present.

Area Type

Figure 51 provides the area type AF.

$$AF_{at,kabc} = \exp\left(0.367 \times I_{at}\right)$$

Figure 51. Equation. Area type KABC AF value.

The base condition is a rural area type. The AF shows an expected 44.3-percent increase in KABC crashes when the interchange area type is urban.

Nearby Adjacent Interchange Gore

Figure 52 provides the nearby adjacent interchange gore AF.

$$AF_{gore,kabc} = \exp\left(0.206 \times I_{gore}\right)$$

Figure 52. Equation. Nearby gore KABC AF value.

The base condition is no adjacent interchange gores within 0.5 mi of the subject interchange's gore locations. The AF shows an expected 22.9-percent increase in KABC crashes when at least one adjacent interchange gore is within 0.5 mi of the subject interchange's gore location.

Managed Lane

Figure 53 provides the managed lane AF.

$$AF_{ml,kabc} = \exp\left(0.282 \times I_{ml}\right)$$

Figure 53. Equation. Managed lane KABC AF value.

The base condition is no managed lanes present within the interchange area. The AF shows an expected 32.6-percent increase in KABC crashes when at least one managed lane is present within the interchange area.

Skew Angle

Figure 54 provides the skew angle AF.

$$AF_{skew,kabc} = \exp\left(0.235 \times I_{skew}\right)$$

Figure 54. Equation. Interchange skew angle KABC AF value.

The base condition is no skew between the freeway mainline and crossroad. The AF shows an expected 26.5-percent increase in KABC crashes when there is a skew angle between the freeway mainline and crossroad that is at least 30 degrees.

Crossroad Left-Turn Lanes

Figure 55 provides the crossroad left-turn lanes AF.

$$AF_{xrlt,kabc} = \exp\left(-0.056 \times LT_{xr}\right)$$

Figure 55. Equation. Crossroad number of left-turn lanes KABC AF value.

The base condition is no left-turn lanes present on the crossroad. The AF is applicable from zero to seven crossroad left-turn lanes. The AF ranges from a 0-percent reduction to a 32.5-percent reduction in KABC crashes when there are seven left-turn lanes combined for all crossroad ramp terminal approaches on crossroad approaches.

Ramp Volume Variability

Figure 56 provides the ramp volume variability AF.

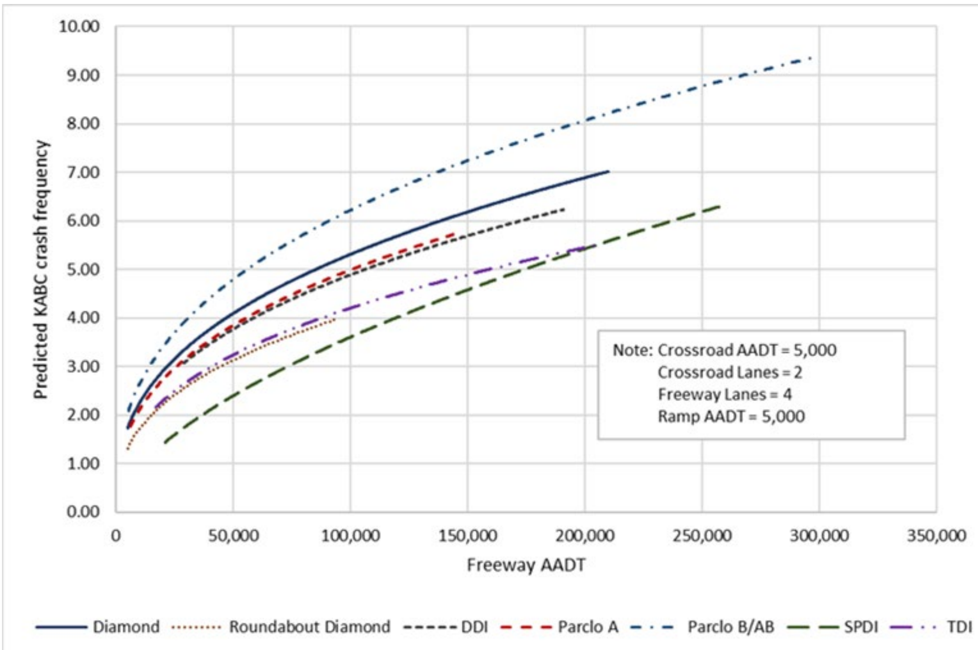
$$AF_{cvrvol,kabc} = \exp\left(-0.299 \times CV_{rvol}\right)$$

Figure 56. Equation. Ramp AADT COV KABC AF value.

The base condition is no variability in ramp volumes. The COV is computed by taking the standard deviation of ramp volumes and dividing by the average ramp volume. The AF is applicable from a COV from 0 to 1.15. The AF ranges from a 0-percent reduction to a 29.1-percent reduction in KABC crashes when the COV is 1.15.

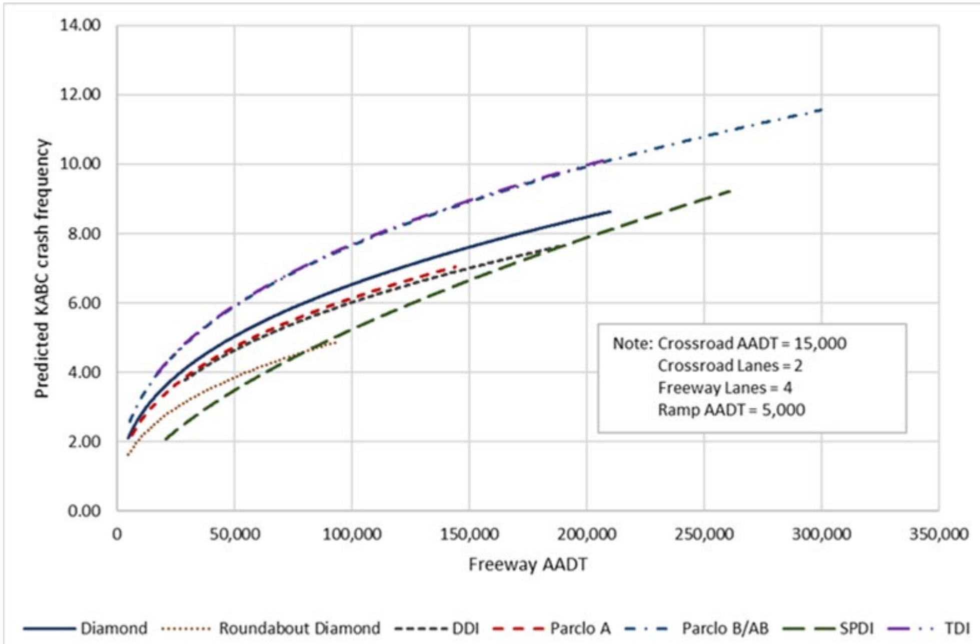
Sensitivity Analysis

The project team conducted a sensitivity analysis for the base SPF for three different scenarios. SPFs are sensitive to freeway AADT, crossroad AADT, ramp AADT, number of freeway through lanes, number of crossroad through lanes, and interchange configuration. The purpose of the sensitivity analysis was to compare base predictions for each interchange configuration given a set of inputs to determine how the input characteristics influence the relative comparison of predicted safety performance. For the first scenario, the project team assumed the freeway had four through lanes, the crossroad had two through lanes and an AADT of 5,000 vehicles per day, and the ramps had a combined total of 5,000 vehicles per day. For the second scenario, the crossroad AADT was changed to 15,000 vehicles per day. For the third scenario, the crossroad AADT was 55,000 vehicles per day (and four travel lanes), and the total combined ramp AADT was 55,000 vehicles per day. Figure 57 through figure 59 plot the predicted crash frequency as a function of freeway AADT, assuming the applicable ranges for each interchange configuration.



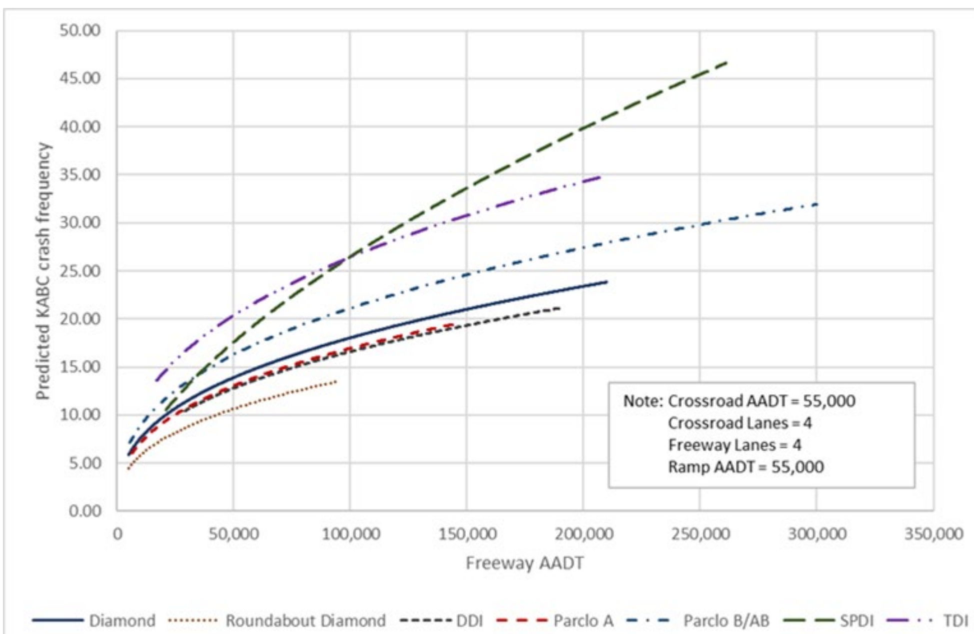
Source: FHWA.

Figure 57. Graph. Predicted KABC crash frequency for scenario 1.



Source: FHWA.

Figure 58. Graph. Predicted KABC crash frequency for scenario 2.



Source: FHWA.

Figure 59. Graph. Predicted KABC crash frequency for scenario 3.

The sensitivity analysis indicates that when crossroad and ramp volumes were lower (in this case, AADT of 5,000 vehicles per day), the SPDI had the lowest predicted crash frequency. This result is at the very low end of the range for which an SPDI or TDI was observed in terms of crossroad and ramp AADTs. The roundabout diamond and TDI had the next lowest predicted KABC crash frequency. In scenario 1, the parclo B or AB had the highest predicted KABC crash frequency.

For scenario 2, where the crossroad AADT was 15,000 vehicles per day, the predicted range of KABC crash frequency narrowed across configurations. Notably, the TDI and parclo B or AB in this scenario had very similar predicted KABC crash frequency at the top of the range. The roundabout diamond was at the low end of the predictive range, similar to SPDI.

For scenario 3, where both the crossroad and total ramp AADTs were 55,000 vehicles per day, the predicted range of KABC crash frequency varied substantially. The roundabout diamond had the lowest predicted crash frequency for its entire applicable range. The TDI and SPDI predicted the highest KABC crash frequency depending on the freeway AADT range.

PDO CRASH FREQUENCY PREDICTION MODEL

This section describes the development of predictive model equations based on PDO crash data. It follows the format of the Fatal and Injury Crash Frequency Prediction Model section and consists of the same five subsections.

Model Development

The proposed predictive model equations and the methods used to develop them were described in the Fatal and Injury Crash Frequency Prediction Model section. These equations were shown in figure 36 through figure 46. In this section, the subscript "kabc" changes to "pdo" in each equation. The base conditions in the KABC crash frequency models are also applicable to the PDO crash frequency model.

For the base SPF, b_{10} applies to a DDI and is interacted with freeway volume per lane multiplied by ramp volume, b_{11} applies to a parclo and is interacted with freeway volume per lane multiplied by ramp volume, and b_{12} applies to a TDI and is interacted with freeway volume per lane multiplied by ramp volume.

Model Estimation

As indicated in the previous section, the project database included all interchange configurations in a pooled dataset. With this approach, the regression model was used to develop the equations in figure 36 through figure 46 from a single equation. The coefficients for variables specific to each interchange configuration are pulled from the model. Variables not specific to an interchange configuration are estimated with data from all configurations together. This approach constrains the impacts of these variables to be the same across interchange configurations.

Table 17 provides the generalized negative binomial regression model results. The final model includes 261 interchanges in the pooled dataset. The model fit statistics indicate an improvement in the overall fit by including the associated predictors.

Table 17. Final predictive model and parameters for PDO crash frequency.

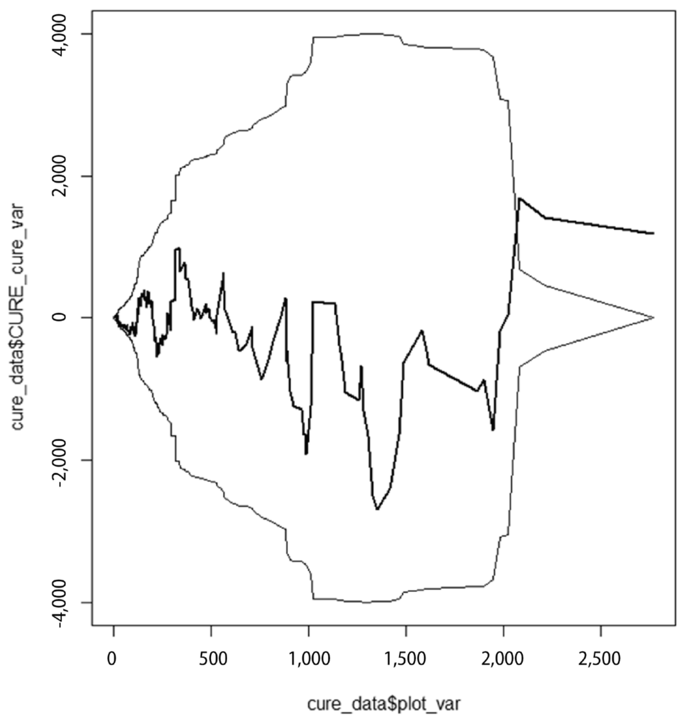
Variable	Description	Value	Standard Error	z-statistic
$b_{0,pdo}$	Interchange constant	−6.642	0.854	−7.78*
$b_{1,pdo}$	Freeway volume per lane interaction with ramp volume	0.415	0.055	7.56*
$b_{2,pdo}$	Crossroad volume per lane	0.215	0.062	3.49*
$b_{3,pdo}$	DDI main effect	3.233	2.894	1.12
$b_{4,pdo}$	Parclo interchange main effect	1.244	1.014	1.23
$b_{5,pdo}$	Parclo type A interchange additive effect	−0.202	0.129	−1.57
$b_{6,pdo}$	SPDI main effect	−4.238	2.127	−1.99*
$b_{7,pdo}$	TDI main effect	−2.918	2.337	−1.25
$b_{8,pdo}$	Roundabout diamond interchange main effect	−0.241	0.160	−1.50
$b_{9,pdo}$	Interaction between SPDI and freeway volume per lane	0.208	0.106	1.95*
$b_{10,pdo}$	Interaction between DDI and freeway volume per lane	−0.177	0.146	−1.21
$b_{11,pdo}$	Interaction between parclo and freeway volume per lane	−0.061	0.054	−1.12
$b_{12,pdo}$	Interaction between TDI and freeway volume per lane	0.142	0.119	1.20
$b_{fr56,pdo}$	Freeway five or six through lanes	0.317	0.099	3.20*
$b_{fr>6,pdo}$	Freeway with more than six through lanes	0.746	0.126	5.90*
$b_{xr>4,pdo}$	Crossroad more than four through lanes	0.195	0.092	2.13*
$b_{at,pdo}$	Urban area type	0.232	0.13	2.05*
$b_{gore,pdo}$	Minimum nearby gore distance within 0.5 mi	0.193	0.129	1.50
$b_{ml,pdo}$	Managed lane presence	0.234	0.143	1.63
$b_{xrlt,pdo}$	Crossroad left-turn lanes	−0.038	0.05	−0.79
$b_{cvrvol,pdo}$	Ramp volume variability	−0.206	0.145	−1.43
$b_{skew,pdo}$	Interchange skew angle	0.117	0.100	1.17

*Significant at 0.05 level.

Overdispersion parameter: 0.260.

Model Evaluation

The project team evaluated the validity of the models through CURE plots to maximize the sample size used for model development. Figure 60 provides the CURE plot for PDO crashes. As shown, the CUREs fall within the confidence intervals for nearly the entire range of predicted crashes. The CUREs generally oscillate about 0, indicating minimum bias for any range of predicted crash frequency. Overall, the CURE plot indicates a good fit between the predicted and observed data.



Source: FHWA.

Figure 60. Graphic. CURE plot for PDO crashes.

Estimated AFs

The project team estimated several AFs in conjunction with the SPFs using PDO crash data. Collectively, they describe the relationship between various factors and crash frequency. This section describes the AFs, including the range of applicability.

Freeway Through Lanes

Figure 61 and figure 62 provide the number of freeway through lanes AFs.

$$AF_{fr\,56,pdo} = \exp\,(0.317 \times I_{fr\,56})$$

Figure 61. Equation. Freeway with five and six lanes PDO AF value.

$$AF_{fr\,>6,pdo} = \exp\,(0.746 \times I_{fr\,>6})$$

Figure 62. Equation. Freeway with more than six lanes PDO AF value.

The base condition is four freeway through lanes. The AF shows an expected 37.3-percent increase in PDO crashes when five or six freeway through lanes are present. The AF shows an expected 110.9-percent increase in PDO crashes when more than six freeway through lanes are present.

Crossroad Through Lanes

Figure 63 provides the number of crossroad through lanes AF.

$$AF_{xr>4,pdo} = \exp\left(0.195 \times I_{xr>4}\right)$$

Figure 63. Equation. Crossroad through lanes PDO AF value.

The base condition is four or fewer crossroad through lanes. The AF shows an expected 21.5-percent increase in PDO crashes when five or more crossroad through lanes are present.

Area Type

Figure 64 provides the area type AF.

$$AF_{at,pdo} = \exp\left(0.232 \times I_{at}\right)$$

Figure 64. Equation. Area type PDO AF value.

The base condition is a rural area type. The AF shows an expected 26.1-percent increase in PDO crashes when the interchange area type is urban.

Nearby Adjacent Interchange Gore

Figure 65 provides the nearby adjacent interchange gore AF.

$$AF_{gore,pdo} = \exp\left(0.193 \times I_{gore}\right)$$

Figure 65. Equation. Nearby adjacent interchange PDO AF value.

The base condition is no adjacent interchange gores within 0.5 mi of the subject interchange's gore locations. The AF shows an expected 21.3-percent increase in PDO crashes when at least one adjacent interchange gore is within 0.5 mi of the subject interchange's gore location.

Managed Lane

Figure 66 provides the managed lane AF.

$$AF_{ml,pdo} = \exp\left(0.234 \times I_{ml}\right)$$

Figure 66. Equation. Managed lane PDO AF value.

The base condition is no managed lanes present within the interchange area. The AF shows an expected 26.4-percent increase in PDO crashes when at least one managed lane is present within the interchange area.

Skew Angle

Figure 67 provides the skew angle AF.

$$AF_{skew,pdo} = \exp(0.117 \times I_{skew})$$

Figure 67. Equation. Interchange skew angle PDO AF value.

The base condition is no skew between the freeway mainline and crossroad. The AF shows an expected 12.4-percent increase in PDO crashes when there is a skew angle between the freeway mainline and crossroad of at least 30 degrees.

Crossroad Left-Turn Lanes

Figure 68 provides the crossroad left-turn lanes AF.

$$AF_{xrlt,pdo} = \exp(-0.038 \times LT_{xr})$$

Figure 68. Equation. Crossroad number of left-turn lanes PDO AF value.

The base condition is no left-turn lanes present on the crossroad. The AF is applicable from zero to seven crossroad left-turn lanes. The AF ranges from a 0-percent reduction to a 23.4-percent reduction in PDO crashes when seven left-turn lanes are combined for all crossroad ramp terminal approaches on crossroad approaches.

Ramp Volume Variability

Figure 69 provides the ramp volume variability AF.

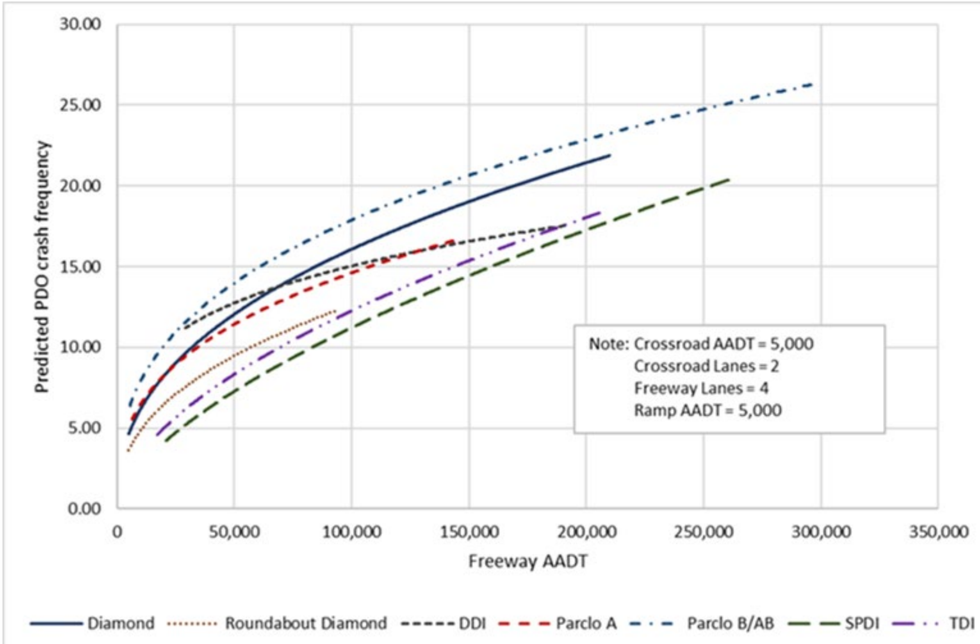
$$AF_{cvrvol,pdo} = \exp(-0.206 \times CV_{rvol})$$

Figure 69. Equation. Ramp AADT COV PDO AF value.

The base condition is no variability in ramp volumes. The COV is computed by taking the standard deviation of ramp volumes and dividing by the average ramp volume. The AF is applicable from a COV from 0 to 1.15. The AF ranges from a 0-percent reduction to a 21.1-percent reduction for PDO crashes when the COV is 1.15.

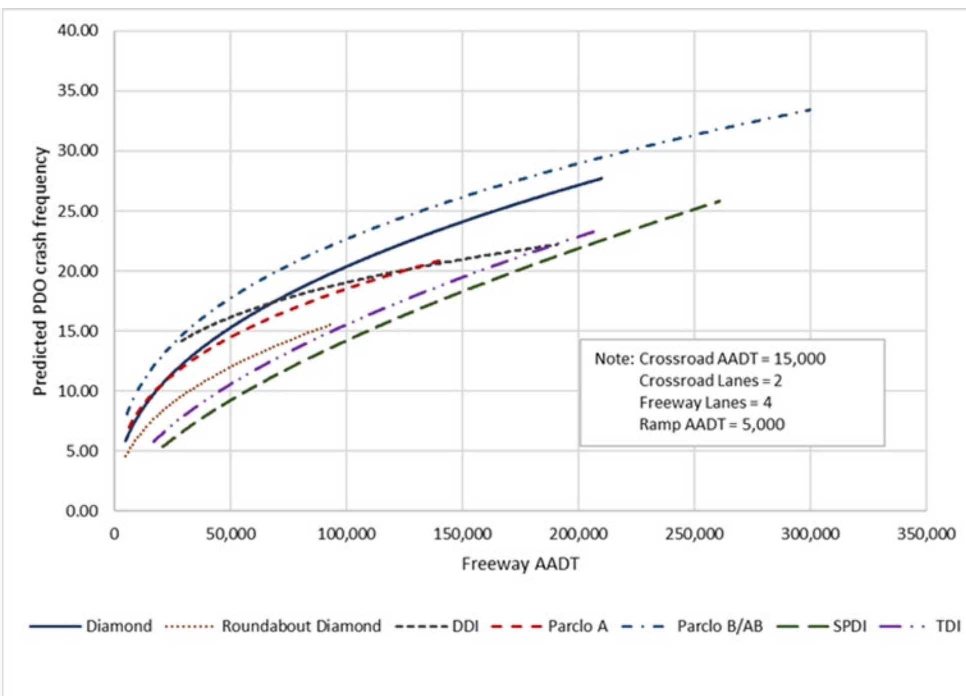
Sensitivity Analysis

The project team conducted a sensitivity analysis for the base SPF for the same three scenarios as in the Fatal and Injury Crash Frequency Prediction Model section. Figure 70 through figure 72 provide the graphical representation of the SPF for the three scenarios. The sensitivity analysis indicates that when crossroad and ramp volumes are lower (in this case, 5,000 vehicles per day AADT), the SPDI had the lowest predicted crash frequency. The roundabout diamond and TDI had the next lowest predicted PDO crash frequency. In scenario 1, the parclo B or AB had the highest predicted PDO crash frequency. These results are consistent with the KABC findings.



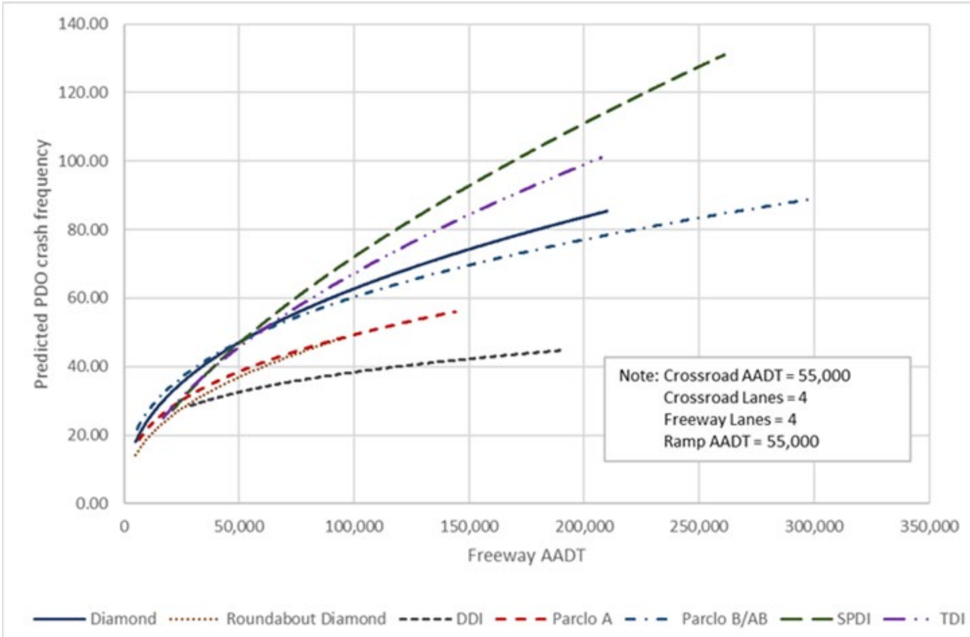
Source: FHWA.

Figure 70. Graph. Predicted PDO crash frequency for scenario 1.



Source: FHWA.

Figure 71. Graph. Predicted PDO crash frequency for scenario 2.



Source: FHWA.

Figure 72. Graph. Predicted PDO crash frequency for scenario 3.

For scenario 2, where the crossroad AADT was 15,000 vehicles per day, the predicted range of PDO crash frequency narrowed across configurations. Notably, the TDI and parclo B or AB in this scenario had very similar predicted KABC crash frequencies at the top of the range. This was not the case for PDO crash frequency. Here, the TDI generally had a lower predicted crash frequency than most other interchange configurations. The SPDI remained the lowest in terms of predicted PDO crash frequency.

For scenario 3, where both the crossroad and total ramp AADTs were 55,000 vehicles per day, the predicted range of PDO crash frequency varied substantially. In this case, the roundabout diamond and DDI had the lowest predicted crash frequencies across the freeway AADT range. The TDI and SPDI predicted the highest PDO crash frequency when the freeway AADT was higher than 60,000 vehicles per day. The parclo B or AB predicted the highest crash frequency when the freeway AADT was less than 60,000 vehicles per day. The results indicate the predicted crash frequency among interchange configurations is highly sensitive to all AADT inputs and ranges. Notably, this sensitivity applies to AADT per lane. For comparative purposes, the project team assumed four freeway lanes. At higher values of AADT, more lanes would be expected; however, the relative comparison across interchange configurations would remain the same since the multiplier for the number of lanes is applied uniformly across configurations. Additionally, since capacity varies among interchange configurations, AADT per lane sensitivity would be important when considering a different number of lanes for the interchange configuration.

CHAPTER 6. SDF DEVELOPMENT

This chapter describes the project team's SDF development for interchange configurations included in this study. The SDF uses a predictive model for calculating the crash severity distribution.

Analysts can use the SDF in conjunction with the KABC crash to predict crash frequency for specific levels of crash severity. Since SDFs include variables based on interchange geometry or operations, SDFs allow for the CPM's severity predictions to be sensitive to specific features of interchanges or configurations.

This chapter consists of two parts: a description of the proposed SDF model development and a description of the model parameters, including an assessment of the impact on crash severity.

BACKGROUND

The project team developed one SDF applicable for all interchange configurations. The model predicts the proportion of crashes by severity level. Specifically, the model predicts the proportion of K, A, B, and C crashes, given that a crash has occurred. In combination, these proportions predict KABC crash frequency disaggregated by individual severity level (in combination with the crash frequency models presented in chapter 5). This approach is intended to minimize the frequency-severity indeterminacy problem described by Hauer and is consistent with the methodology used for the freeway safety prediction model in chapter 18 of the HSM Supplement.[1]

DATABASE SUMMARY

This summary provides details on the range of data included in the database and provides some insight into the development of the predictive model form. The project team drew the data presented in this chapter from the same dataset used to develop the crash frequency models; however, the observational unit in this dataset is an individual crash rather than an interchange.

The project team captured severity data for all five States included in the crash frequency dataset, which allowed them to include all States in the SDF database. The following sections provide an overview of the geometric and operational characteristics as well as the crash characteristics included in the SDF dataset.

Geometric and Operational Characteristics

Table 18 summarizes the geometric and operational characteristics of the sites in the SDF database. The pooled dataset includes crash data from all interchange configurations in the crash frequency dataset from all five States.

Table 18. Summary of SDF dataset geometric and operational characteristics.

Variable	Average	Minimum	Maximum
Intersection skew angle (degrees)	12.27	0	60
Freeway through lanes (both directions) (No.)	7.28	4	12
Crossroad through lanes (both directions) (No.)	4.41	2	6
Freeway AADT (No.)	132,054	5,028	300,000
Crossroad AADT (No.)	29,112	168	68,000
Freeway posted speed limit (mph)	61.95	35	75
Crossroad posted speed limit (mph)	38.37	25	65
Interchange length along freeway (miles)	0.51	0.1	1.15
Interchange length along crossroad (miles)	0.16	0.07	0.59
Crossroad ramp terminal separation (miles)	0.10	0	0.80
Adjacent crossroad minimum distance (miles)	1.82	0.36	9.65
Adjacent crossroad average distance (miles)	1.52	0.51	10.45
Adjacent gore minimum distance (miles)	0.73	0.13	8.97
Adjacent gore average distance (miles)	1.01	0.17	9.74
Adjacent intersection minimum distance (miles)	15.17	0	550
Adjacent intersection average distance (miles)	18.82	0.05	710
Combined exit ramp length (miles)	0.59	0.19	1.46
Combined entrance ramp length (miles)	0.56	0.20	1.91
Combined entrance ramp AADT (No.)	18,946	33	69,985
Combined exit ramp AADT (No.)	18,960	15	74,985
Driveway access points (No.)	0.32	0	7
Maximum lanes crossed by pedestrians (No.)	4.57	0	10
Pedestrian right-turn conflicts (No.)	0.56	0	7

Table 19 provides an overview of the categorical data for sites included in the SDF database. Note that while the same data are included in the crash frequency database, the data in table 18 and table 19 only include KABC crashes. In addition to differences in safety performance across locations, these data yield different averages and distributions of input elements for the crash-based dataset from the interchange-based dataset.

Table 19. Summary of SDF information for categorical data.

Category	Characteristic	Frequency (percent)
Nonramp leg at terminal	Present	1.83
	Not present	98.17
Crossroad ramp terminal control	All-way stop	0.13
	Ramp stop	5.48
	Roundabout	4.13
	Signal	90.25
	Yield	<0.01
Area type	Rural	6.14
	Urban	93.86
Crossroad location	Overpass	50.82
	Underpass	49.18
Weaving section at interchange area boundary	Present	24.69
	Not present	75.31
Frontage road	Present	11.18
	Not present	88.82
Sidewalk	Present	69.02
	Not present	30.98
Lighting	Present	96.30
	Not present	3.70
Freeway median type	Rigid barrier	76.63
	Semirigid barrier	5.81
	Flexible barrier	7.96
	Depressed/grass	9.60
Collector distributor road	Not present	93.26
	Present	6.74
Managed lane	Not present	70.65
	Present	29.35

Crash Characteristics

The project team translated 5 yr of crash data from the interchange database to the SDF database (unless data were removed due to construction). The study period included 2016–2020 for all States.

Table 20 provides an overview of the crash severity database by State and presents the data by individual severity level as well as for fatal and injury crashes combined (due to the sample size of fatal crashes (K) being fewer than 100 crashes per year). There is some variability among States; however, this variability is expected due to differences in reporting characteristics and interchange configurations and design across States. Using K and A crashes combined allowed the project team to have a larger sample in each category for SDF development. Table 21 presents the crash severity data by interchange configuration.

Table 20. Summary of crash severity data by State.

State	K	A	B	C	KA	Total KABC
Arizona	44	231	2,345	3,048	275	5,668
Missouri	184	396	677	14,283	580	15,540
North Carolina	38	78	583	1,930	116	2,629
Ohio	14	84	685	764	98	1,547
Utah	34	199	1,242	3,650	233	5,125
Combined	314	988	5,532	23,675	1,302	30,509

Table 21. Summary of crash severity data by interchange configuration.

Interchange Configuration	K	A	B	C	KA	Total KABC
Diamond	44	103	420	1,572	147	2,139
CD	20	52	437	737	72	1,246
TDI	41	128	843	3,540	169	4,552
DDI	58	134	381	3,464	192	4,037
Parclo A	9	64	290	1,342	73	1,705
Parclo B	22	52	227	998	74	1,299
Parclo AB	47	143	545	2,832	190	3,567
SPDI	73	312	2,389	9,190	385	11,964
Combined	314	988	5,532	23,675	1,302	30,509

SDF Model Development

This section describes the steps taken to develop the SDFs. The following subsections provide a description of the model form, an overview of the modeling approach, an overview of the statistical analysis methods, and a discussion of the findings.

Predictive Model Form

The severity of outcomes is defined by the probability of the outcome occurring. Figure 73 defines the probability of outcome *m*.

$$P_m = \frac{e^{V_m}}{\sum_{i=1}^{M} e^{V_i}}$$

Figure 73. Equation. Probability of outcome *m*.

Where:

P_m = probability of the outcome *m*.
M = total number of possible outcomes modeled.

V_i = deterministic component of outcome i.
V_m = deterministic component of outcome m.

In this case, the SDFs represent injury crash outcomes (K, A, and B), with C as the base scenario. Initial results indicated that crash sample sizes for fatalities (i.e., K) were small; therefore, K and A were combined for fatal and severe-injury crashes (KA). Analysts can differentiate K and A crashes through a proportion factor from the observed dataset. The probability for each outcome is shown in figure 74 through figure 77.

$$P_{KA} = \frac{e^{V_{KA}}}{\frac{1}{C_{sdf}} + e^{V_{KA}} + e^{V_B}}$$

Figure 74. Equation. Probability of KA severity outcome.

Where:
P_{KA} = probability of KA severity outcome.
V_B = deterministic component of B severity outcome.
V_{KA} = deterministic component of KA severity outcome.

$$P_B = \frac{e^{V_B}}{\frac{1}{C_{sdf}} + e^{V_{KA}} + e^{V_B}}$$

Figure 75. Equation. Probability of B severity outcome.

$$P_C = 1.0 - (P_{KA} + P_B)$$

Figure 76. Equation. Probability of C severity outcome.

with

$$V_m = V_{m,b} \times f_{m,1} \ldots \times f_{m,n}$$

Figure 77. Equation. Deterministic component of outcome *m*.

Where:
C_{sdf} = local calibration factor.
$f_{m,i}$ = severity AF for the relationship between severity m of traffic characteristics, geometric element, or traffic control feature i (i=to n).
n = total number of severity AFs.
$V_{m,b}$ = base deterministic component of outcome m.

The deterministic component is a dimensionless number indicating the relative frequency of crashes associated with a specific severity level (i.e., K, A, B, or C), given that a crash has occurred. Severity level C is calculated as a total probability of 1.0 minus the assigned probabilities for KA- and B-level severities. The value C_{sdf} is the local calibration factor agencies can use to calibrate the model to their specific locations. The calibration procedure is provided in the HSM Part C, appendix A.[3]

Modeling Approach

Pooled Dataset Approach

As noted in the Crash Characteristics section, the project team opted to combine the SDF datasets into one pooled dataset since the sample proportions were relatively similar between interchange configurations. Additionally, this step was necessary due to small samples for more severe crashes for individual interchange configurations. The project team used the pooled dataset to estimate one regression model and used data for all interchange configurations. They then applied the coefficients to develop an SDF for each interchange configuration. As with the crash frequency models, some factors can be specific to interchange configuration, while others are common to all SDFs. The project team could then maximize the use of the sample dataset, particularly for individual interchange configurations with smaller sample sizes.

Model Development Process

The project team used a similar approach to the crash frequency modeling approach, prioritizing characteristics previously found to be related to crash severity for inclusion in the SDF. The project team added independent variables one at a time based on statistical significance, magnitude and direction of effect, and impact on other variables in the model. Since they used C (possible injury) crashes as the baseline, the project team entered variables for combined KA- and B-level injuries simultaneously. The magnitude and direction of effect for each severity category relative to the baseline and to each other were assessed. If the results were intuitive, the project team retained the variables. If the results indicated little to no difference in effect across adjacent severity categories, the project team constrained the coefficients to be the same. Otherwise, if the effect was not statistically or practically significant, the variable was dropped from the specific severity-level category.

Statistical Analysis Method

The project team used a multinomial logit model to analyze crash severity. This model allows for some variables to be constrained to have the same effect on each severity level while allowing other variables to have a fluctuating effect among levels. An investigation of the reliability of this approach has found that it tends to outperform the direction calibration of SPFs for each severity level.[12] This approach has been successfully applied to develop freeway predictive methods for the HSM.[7,18] For this research, the baseline is C-level injury crashes (but also considering combined K and A), and individual severity proportions are evaluated based on predictor variables.

SDF Prediction Model

Model Development

This subsection describes the regression model and the project team's methods for estimating the model's coefficients. The regression model is generalized to accommodate each interchange configuration. The generalized form shows the variables included in the model. For some variables, the project team used indicator variables to determine when the corresponding AF was applicable.

Figure 78 through figure 94 describe the regression model the project team calibrated using the severity data.

$$P_m = \frac{V_m}{1 + V_{KA} + V_B}$$

Figure 78. Equation. Projected probability of severity outcome *m*.

$$V_{KA} = V_{b,KA} \times f_{AADTfr,KA} \times f_{AADTxr,KA} \times f_{fr65,KA} \times f_{xr45,KA} \times f_{PAorB,KA} \times f_{PAB,KA} \times f_{SPDI\_DDI\_TDI,KA} \times f_{RDI,KA} \\ \times f_{INT10,KA} \times f_{GORE25,KA} \times f_{PED\_RT,KA} \times f_{fr\_8l,KA} \times f_{xr\_4l,KA}$$

Figure 79. Equation. Projected deterministic component of KA crash severity.

$$V_B = V_{b,B} \times f_{fr65,B} \times f_{xr45,B} \times f_{PAorB,B} \times f_{PAB,B} \times f_{SPDI\_DDI\_TDI,B} \times f_{RDI,B} \times f_{INT10,B} \times f_{PED\_RT,B} \times f_{fr\_8l,B} \times f_{xr\_4l,B}$$

Figure 80. Equation. Projected deterministic component of B crash severity.

$$V_{b,m} = \exp\left[b_{0,m}\right]$$

Figure 81. Equation. Constant value for outcome *m*.

$$f_{AADTfr,KA} = \exp\left[b_{AADTfr,KA} \times I_{fr200k}\right]$$

Figure 82. Equation. Freeway AADT severity AF.

$$f_{AADTxr,KA} = \exp\left[b_{AADTxr,KA} \times I_{xr30k}\right]$$

Figure 83. Equation. Crossroad AADT severity AF.

$$f_{fr65,m} = \exp\left[b_{fr65,m} \times I_{fr65}\right]$$

Figure 84. Equation. Freeway posted speed limit severity AF.

$$f_{xr45,m} = \exp\left[b_{xr45,m} \times I_{xr45}\right]$$

Figure 85. Equation. Crossroad posted speed limit severity AF.

$$f_{PAorB,m} = \exp\left[b_{PAorB,m} \times I_{PAorB}\right]$$

Figure 86. Equation. Parclo type A or B severity AF.

$$f_{PAB,m} = \exp\left[b_{PAB,m} \times I_{PAB}\right]$$

Figure 87. Equation. Parclo type AB severity AF.

$$f_{SPDI\_DDI\_TDI,m} = \exp\left[b_{SPDI\_DDI\_TDI,m} \times I_{SPDI\_DDI\_TDI}\right]$$

Figure 88. Equation. SPDI, DDI, or TDI severity AF.

$$f_{RDI,m} = \exp\left[b_{RDI,m} \times I_{RDI}\right]$$

Figure 89. Equation. Roundabout diamond severity AF.

$$f_{INT10,m} = \exp\left[b_{INT10,m} \times I_{INT10}\right]$$

Figure 90. Equation. Nearby adjacent intersection severity AF.

$$f_{GORE25,KA} = \exp\left[b_{GORE25,KA} \times I_{GORE25}\right]$$

Figure 91. Equation. Nearby adjacent interchange gore severity AF.

$$f_{PED\_RT,m} = \exp\left[b_{PED\_RT,m} \times N_{PED\_RT}\right]$$

Figure 92. Equation. Pedestrian right-turn conflict severity AF.

$$f_{fr\_8l,m} = \exp\left[b_{fr\_8l,m} \times I_{fr\_8l}\right]$$

Figure 93. Equation. Number of freeway lanes severity AF.

$$f_{xr\_4l,m} = \exp\left[b_{xr\_4l,m} \times I_{xr\_4l}\right]$$

Figure 94. Equation. Number of crossroad lanes severity AF.

Where:

b_i = regression coefficient for condition i.

$V_{b,m}$ = base deterministic component for severity m.

$f_{AADTfr,KA}$ = AF for the relationship between freeway bidirectional AADT greater than or equal to 200,000 vehicles per day and KA severity.

$f_{AADTxr,KA}$ = AF for the relationship between crossroad AADT greater than or equal to 30,000 vehicles per day and KA severity.

$f_{fr65,m}$ = AF for the relationship between freeway posted speed limit greater than or equal to 65 mph and severity m.

$f_{fr\_8l,m}$ = AF for the relationship between freeways with eight or more lanes and severity m.

$f_{GORE25,KA}$ = AF for the relationship between adjacent gore separation less than 0.25 mi and KA severity.

$f_{INT10,m}$ = AF for the relationship between adjacent intersection spacing being less than 0.10 mi and severity m.
$f_{PAB,m}$ = AF for the relationship between parclo type AB and severity m.
$f_{PAorB,m}$ = AF for the relationship between parclo type A or B and severity m.
$f_{PED\_RT,m}$ = AF for the relationship between number of pedestrian crossings conflicting with right turns and severity m.
$f_{RDI,m}$ = AF for the relationship between roundabout diamond and severity m.
$f_{SPDI\_DDI\_TDI,m}$ = AF for the relationship between SPDI, DDI, or TDI and severity m.
$f_{xr45,m}$ = AF for the relationship between crossroad posted speed limit greater than or equal to 45 mph and severity m.
$f_{xr\_41,m}$ = severity AF for the relationship between crossroads with four or more lanes and severity m.
I_{fr200k} = indicator for freeway bidirectional AADT greater than or equal to 200,000 vehicles per day.
I_{fr65} = indicator for freeway posted speed limit greater than or equal to 65 mph.
I_{fr_81} = indicator for freeway having eight or more through lanes.
I_{GORE25} = indicator for nearest adjacent interchange gore within 0.25 mi.
I_{INT10} = indicator for nearest adjacent intersection within 0.10 mi.
I_{PAB} = indicator for the presence of a parclo type AB interchange.
I_{PAorB} = indicator for the presence of a parclo type A or B interchange.
I_{RDI} = indicator for the presence of a roundabout diamond interchange.
$I_{SPDI\_DDI\_TDI}$ = indicator for the presence of an SPDI, DDI, or TDI interchange.
I_{xr45} = indicator for crossroad posted speed limit greater than or equal to 45 mph.
I_{xr_41} = indicator for crossroad having four or more lanes.
I_{xr30k} = indicator for crossroad bidirectional AADT greater than or equal to 30,000 vehicles per day.
N_{PED_RT} = number of pedestrian crossings conflicting with right turns.

Model Estimation

Table 22 presents the results of the SDF regression model. The table provides the regression coefficient and standard error for each variable by severity level in the resulting model. The z-statistic (for assessing statistical significance) was computed by dividing the coefficient by the standard error. As evidenced by the large sample size, many factors fit into the final SDF for combined KA and B crash severities. The project team focused on including factors differentiating the effects of interchange configuration. The base condition for this model is a C-level injury.

Table 22. Summary of pooled data from SDF model.

Variable	KA Coefficient	KA Standard Error	B Coefficient	B Standard Error
$b_{0,m}$	−3.104*	0.078	−1.956*	0.067
$b_{AADTfr,KA}$	−0.786*	0.098	N/A	N/A
$b_{AADTxr,KA}$	−0.177*	0.063	N/A	N/A
$b_{fr65,m}$	0.870*	0.034	0.870*	0.034
$b_{xr45,m}$	0.231*	0.036	0.231*	0.036
$b_{fr81,m}$	0.483*	0.033	0.483*	0.033
$b_{xr41,m}$	0.177*	0.051	0.177*	0.051
$b_{GORE25,KA}$	0.302*	0.079	N/A	N/A
$b_{INT10,m}$	1.230*	0.251	1.230*	0.251
$b_{PED\_RT,m}$	0.025	0.015	0.025	0.015
$b_{PAB,m}$	−0.510*	0.062	−0.510*	0.062
$b_{PAorB,m}$	−0.446*	0.067	−0.446*	0.067
$b_{RDI,m}$	−0.848*	0.093	−0.848*	0.093
$b_{SPDI\_DDI\_TDI,m}$	−0.745*	0.075	−0.518*	0.053

*Indicates significant at 0.05 level.

The coefficients fit into figure 81 through figure 94 as indicated by the applicable variable. In some cases, variables apply to specific interchange configurations, while others are generalizable to all interchange configurations.

The results in table 22 indicate generally consistent effects for variables significantly associated with KA and B crash severities. In other words, the coefficients for KA and B severity crashes were the same in most cases. The coefficient differed for the indicator for an SPDI, DDI, or TDI configuration. In that case, the coefficient indicated a larger reduction in the likelihood of a KA crash than a B crash compared to a C-crash baseline.

The results indicated freeway volumes greater than or equal to 200,000 vehicles per day were associated with a lower proportion of KA crash severity. The same was true for crossroad volume greater than or equal to 30,000 vehicles per day. The presence of an adjacent interchange gore within 0.25 mi of the subject interchange's gore was associated with a higher probability of a KA crash severity. The model indicated that, compared to a baseline diamond or CD interchange, parclo AB, parclo A or B, roundabout diamond, SPDI, DDI, and TDI interchanges are associated with a lower probability of KA and B crash severities. The other factors indicated an increased probability of KA and B crash severities.

While the project team estimated KA crash probabilities together as part of the SDF, KA crash probability may be split, assuming 24.1 percent of KA crashes are fatal.

CHAPTER 7. CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

This project aimed to develop a planning-level safety prediction model to assess the predicted safety performance of interchange configurations to support IJRs. The project team developed the safety prediction model in a manner consistent with those developed for the HSM.[1] Specifically, the approach is consistent with that found in chapter 18 of the HSM Supplement. The CPM predicts KABC and PDO crash frequency separately at the interchange level. Additionally, the predictive method includes planning-level SDFs to assess the impact of features on crash severity.

The project team examined interchange types accounting for more than 75 percent of those considered in IJRs. This project included the following interchange configurations:

- Diamond.
- CD.
- TDI.
- DDI.
- Roundabout diamond.
- SPDI.
- Parclo type A.
- Parclo type B.
- Parclo type AB.

The predictive method found no safety performance difference between a diamond and CD interchange. These two configurations combined can be assumed as the base condition. The predicted crash frequency (for both KABC and PDO crashes) for interchange configurations relative to the base condition depends on the freeway volume, crossroad volume, combined ramp volumes, number of freeway lanes, and number of crossroad lanes.

The predictive method adjusts for area type, interchange skew, the presence of a nearby interchange, the presence of managed lanes, variability in ramp volumes, and the number of left-turn lanes on the crossroad.

For interchange crash severity, the base condition continued to be a diamond or CD interchange configuration. Crash severity was associated with interchange configuration; however, the project team found a similar severity distribution for SPDI, DDI, and TDI configurations.

The SDFs further identified crash severity proportion to be associated with freeway AADT, crossroad AADT, freeway posted speed limit, crossroad posted speed limit, the presence of a nearby adjacent interchange, the presence of a nearby adjacent intersection on the crossroad, the number of pedestrian crossings conflicting with right-turning vehicles, number of freeway through lanes, and the number of crossroad through lanes.

The project team developed a basic implementation spreadsheet to support testing the relationships found in the CPM. The implementation spreadsheet does not identify if inputs are valid or within the range of data used to develop the predictive models. The implementation spreadsheet includes a prediction for all configurations simultaneously. The user can provide the same inputs for all configurations or vary input parameters for each configuration when conducting a simultaneous test.

Additionally, the implementation spreadsheet provides a confidence interval. The variance for the confidence interval is calculated as shown in figure 95.

$$Var(\lambda_i) = E(\lambda_i)[1 + \alpha \times E(\lambda_i)]$$

Figure 95. Equation. Variance equation for the predictive model.

Where:

$Var(\lambda_i)$ = variance of crashes at the subject interchange.
$E(\lambda_i)$ = expected crash frequency at the subject interchange.
α = overdispersion parameter.

The resulting variance for each prediction yielded comparative results that were not statistically different from each other in all cases. The 95-percent confidence interval typically encompasses a range from approximately zero to slightly more than double the predicted crash frequency for the interchange configuration prediction of interest. Therefore, even though there are practical implications of the CPM from one interchange configuration to another for a given set of input data, the results are not statistically significant.

Practitioners should exercise caution when making decisions based purely on safety prediction models. The purpose of the predictive model is to provide an estimate of safety performance, which should be considered along with other factors, such as cost, operational performance, and environmental constraints. Additionally, CPMs predict the long-term crash history of a location, given a set of input variables. Predictions from these models are more reliable when considering long-term crash frequency and additional factors that may make the location unique. Professional judgment should be exercised if interchange features are atypical or are outside the bounds of the data used to develop the models. Analysts should consult the database summary provided in chapter 4 to determine the bounds of data used to develop this CPM.

RECOMMENDATIONS FOR FUTURE EFFORTS

The predictive method developed in this research applies only to those interchange configurations described in chapters 2 and 3. The project team captured data for interchanges meeting the standard definitions of those configurations. For example, the project team did not capture data for sites with extra ramps, missing ramps, or direct-connection ramps. Future research should consider incorporating the safety effects of those features that make interchanges unique, as planners often look for unique solutions to operational or safety concerns.

Future efforts should also consider expanding the dataset to more sites for analysis. Some configurations had smaller sample sizes, such as parclo B interchanges. Increasing the sample size for each interchange configuration will improve the reliability of the CPM. The project team focused on collecting a broad set of attributes for inclusion in the predictive method. Future efforts could reduce the number of attributes considered to those most closely related to safety and focus on collecting data at more locations.

Additionally, the predictive models should be expanded to include more interchange configurations. For example, double roundabout or dogbone interchanges have become more common, and there should be a sufficient sample to include their safety effects in the predictive model. Future work should also examine the role of signing and markings consistent with roadway design and lane configurations at exits, entrances, and reductions within and between interchanges. This project attempted to assess the effects of lane additions and lane drops, but the data did not include lengths of auxiliary lanes or associated markings and signage.

APPENDIX A: SURVEY PROVIDED TO DIVISIONS

Appendix A includes the survey provided to FHWA divisions to assess how many IARs are submitted per year and which interchange configurations were most commonly considered in IARs. Further, this survey distinguishes between systems and service interchanges.

I. Background: FHWA is seeking to develop planning-level models and tools to predict crash frequency and severity for an existing or proposed service interchange. Model inputs will be limited to details that are generally known at the planning and conceptual-design level and are expected to affect crash frequency and severity. The planning-level models will allow analysts to compare the potential safety performance effects of freeway access and interchange design decisions at the planning level.

   Since the majority of IARs focus on a few specific interchange types, the scope of the planning-level models will focus on those that are most common. The purpose of this survey is to identify those subsets of interchange types that comprise at least 75 percent of those considered in IARs.

II. Introductory question fields:

   A. Name.
   B. Job title.
   C. Email address.
   D. State.

III. General information about requests:

   A. How many IARs does your State typically submit per year (see figure 96 for interchange configurations)?

      a. 0–5.
      b. 6–10.
      c. 11–15.
      d. 16–20.
      e. More than 20.

   B. Of those IARs, how many involve system interchanges (interchanges providing access from one freeway to another)?

      a. 0–2.
      b. 3–5.
      c. 6–8.
      d. More than 8.

   C. Can you estimate the percentage of service interchange IARs that are for rural interchanges (as opposed to urban/suburban interchanges)?

a. 0–25 percent.
b. 26–50 percent.
c. 51–75 percent.
d. More than 75 percent.

D. Approximately how many service interchange IARs per year involve a new interchange?

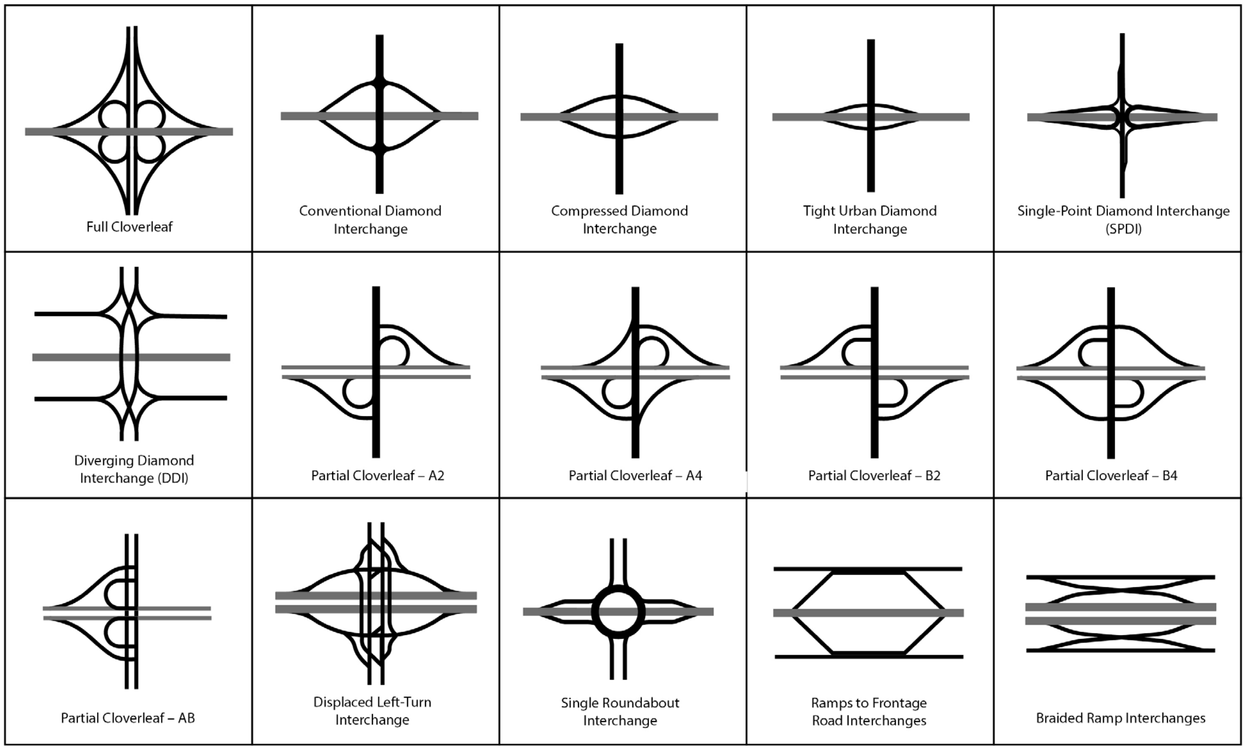
a. 0–5.
b. 6–10.
c. 11–15.
d. More than 15.

E. Approximately how many service interchange IARs per year involve a reconfiguration of an existing interchange?

a. 0–5.
b. 6–10.
c. 11–15.
d. More than 15.

F. Approximately how many service interchange IARs per year involve minor changes (such as modifying ramp terminal access control, adding lanes to an existing ramp, or relocating a ramp terminal to a different roadway)?

a. 0–5.
b. 6–10.
c. 11–15.
d. 16–20.
e. More than 20.



Source: FHWA.

Figure 96. Graphic. Interchange configurations considered in the survey.

G. Considering the interchange configuration provided in the figure 96 (note multiple examples of parclo configurations are provided for informative purposes, but many more variations exist in each category), please select from the list below (table 23) those interchange configurations that have been *proposed* in the IARs your office has reviewed within the last 3–5 yr.

Table 23. Annual IARs reviews by interchange configuration.

Interchange Configuration	Uncommon or Rarely Considered	Sometimes Considered but Rarely Selected	Commonly Considered and Selected 0-2 Times Per Yr	Commonly Considered and Selected 3-5 Times Per Yr	Commonly Considered and Selected 6-8 Times Per Yr	Commonly Considered and Selected More Than 8 Times Per Yr
CLF interchange	—	—	—	—	—	—
Conventional diamond interchange	—	—	—	—	—	—
CD interchange	—	—	—	—	—	—
Tight urban diamond interchange	—	—	—	—	—	—
Single-point urban interchange (SPUI)	—	—	—	—	—	—
DDI	—	—	—	—	—	—
Parclo–A (A2, A4, or variant)	—	—	—	—	—	—
Parclo–B (B2, B4, or variant)	—	—	—	—	—	—
Parclo–AB (or variant thereof)	—	—	—	—	—	—
DLT interchange	—	—	—	—	—	—
SR	—	—	—	—	—	—
Ramps to frontage road interchanges	—	—	—	—	—	—
Braided ramp interchanges	—	—	—	—	—	—

—Empty cell for the respondent to check.

H. If any interchange type(s) not listed in question 12 has been considered or selected in the last 3–5 yr, please list those type(s) here (and please provide an indication of how commonly considered or selected as with question 12).

I. Does your State consider or select roundabouts as ramp terminals for appropriate interchange configurations listed in question 12?

a. Does not commonly consider roundabouts.
b. Commonly considers roundabouts but has not installed many.
c. Commonly installs roundabouts as ramp terminals at appropriate interchange configurations.

J. Does your State consider the use of collector-distributor roads at interchanges?

IV. Followup:

A. Would you be willing to provide more details to the research team through a followup phone call?

B. Can you provide the name, phone number, and email address of a State DOT contact who can provide details on available ramp traffic volume data?

C. Is there anything else you would like to tell us about IARs in your State?

APPENDIX B: ADDITIONAL SURVEY COMMENTS

Table 24 provides additional insights on IJRs (also known as IARs) for various States as indicated by respondents. In general, several respondents indicated that IJRs are seldom received in their States. Several States also indicated IJRs are not required for minor changes, and one State (Washington) noted that nontraditional interchange configurations are more commonplace. Additionally, several respondents specifically called attention to DDIs and single-point urban interchanges in their responses as something they would like to see in the model.

Table 24. Additional comments on IJR (IAR) practices.

State	Is there anything else you would like to tell us about IARs in your State?
Alabama	The type of interchange varies on project constraints. Separation of ramp termini is sometimes an issue, with the State asking to have multiple merge points. Roundabouts are becoming more common, but only using Highway Safety Improvement Program funding for most of them; they are not considered routinely.
Alaska	I do not have much data about our access requests for the past 5 yr as I am new to the office, and we haven’t had any since I started.
California	We are working with the State to develop a standard operating procedure (SOP) and guidelines on how to perform and submit IARs to FHWA. Our office updated our own SOP now that the State has adopted the two-point policy. Also, there are minor activities like adding a lane to a ramp, metering a ramp, or shifting the exit ramp to the same roadway that has been delegated to Caltrans, and we do not see.
Colorado	We use a Minor Interchange Modification Request process for interchanges modifications that do not meet the requirement of needing an IAR.
Delaware	Due to limited interstate length in Delaware, we do not have very many interstate change requests, as a rule.
District of Columbia	IARs are few and far between here in the District of Columbia.
Florida	Florida is operating under a programmatic agreement with the Florida DOT where the majority of interchange access review and approvals are delegated to the State. Therefore, we are less involved with interchange designs.
Hawaii	Hawaii has not proposed any new interchanges in the past 3 yr.
Indiana	A few of the recent IARs have been handled through our programmatic review/approval process where the State does the review and determination of engineering/operational/safety acceptability.
Iowa	Iowa has a process that distinguishes between IJRs and Interchange Operations Reports (IORs). IORs document minor changes at interchanges that do not alter the interchange type or the number and type of access points. For this survey, Iowa’s IJRs were considered the equivalent of IAR’s discussed in the survey.
Kansas	Probably half of ours are modifications to existing interchanges. We have done several DDI retrofits, and we no longer require a full-blown [IAR] for those. Also, the same for roundabouts at the terminals; we don’t require full-blown [IAR]. If I would offer any advice for someone reviewing a [IAR] submittal, I would focus on operations. Safety is very closely tied to operations (i.e., poor operations leads to poor safety). I would also stress with everyone that even though the 20-yr design life is no longer a requirement, best practice should lead engineers to determine what year your level of service will begin to see [level of service] Fs and explore what future options there will be to improve it.
Maryland	They are called Interstate Access Point Approval.

State	Is there anything else you would like to tell us about IARs in your State?
Massachusetts	New interchanges are exceptionally rare. Modifications tend toward the minor, such as signalizing ramp terminals, extending acceleration lanes, or providing ramp widening to add turn lanes.
Michigan	We are hopeful that DDIs and SPUIs will be given consideration.
Nebraska	Very few IARs have been conducted in Nebraska in the past 5 yr.
New Jersey	In New Jersey, IARs recently submitted have primarily involved only modifications to existing interchanges. Configurations identified in question 12 represent existing and/or modified configurations.
New York	The New York State DOT typically only submits 0–5 IARs per year for our review and approval.
North Dakota	North Dakota does not fit your survey very well. FHWA receives about one IAR every other year. In the past 6 yr, IARs in North Dakota have only addressed new interchanges, not modifications to existing interchanges. There have been two IARs in the past 6 yr, one a modified SPUI that does not fit your matrix and a locked gate access with median crossovers for use in emergency flood conditions.
Pennsylvania	We like diverging diamonds and roundabouts at terminals. They work well. We only process a few per year, and most are modifications to existing facilities. Our oversight of toll road interchanges is still a little vague.
Rhode Island	In the past 5 yr, I recall only four IARs. Only one involved a new full interchange between an interstate and local road, and it was a (compact?) diamond interchange. The three others involve (currently in progress) modifying existing interstate interchanges, only one of which is an Interstate system-level interchange modification. These three involve improvements to existing partial interchanges via the addition of and/or modification of partial interchange configurations.
Vermont	We (the Vermont FHWA Division Office) do not get very many IARs. I do not know of any in the 13 yr I have been in the Vermont FHWA Division Office.
Washington	The last four interchanges involved a three-level service interchange, a system-to-system three-level interchange, a half-diamond partial interchange, and a direct access high-occupancy vehicle Texas T. I use this as an example to demonstrate that nontraditional configurations are, in my limited experience, more the commonplace than anything else in the State of Washington.
Wyoming	The Wyoming DOT (WYDOT) rarely designs/modifies a service interchange—less than one per year. WYDOT is currently designing a new system interchange (I–80/25) with a new service interchange, but funding will likely cause the project to be put indefinitely on hold.

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