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Calibration of Highway Safety Manual Prediction Models for Freeway Segments, Speed-Change Lanes, Ramp Segments, and Crossroad Ramp Terminals in Kansas

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16 Abstract

Crash prediction models in the Highway Safety Manual (HSM) are used to quantify the safety experience of new and existing roadways. Safety Performance Functions (SPFs) or crash prediction models are statistical formulas developed on limited data from a few selected states, Kansas not being one of those states. Therefore, the HSM recommends calibration of HSM-default SPFs, or development of local SPFs, to enhance the accuracy of predicted crash frequency. This report demonstrates the HSM calibration procedure and its quality assessment for freeway segments, speed-change lanes, ramp segments, and crossroad ramp terminals in Kansas. The study used three years of recent crash data, the most recent geometric data, and HSM-recommended sample sizes for all facilities considered for the calibration.

The HSM methodology overpredicted fatal and injury (FI) crashes and underpredicted all property damage only (PDO) crashes for freeway segments. The HSM methodology consistently underpredicted both FI and PDO crashes for both entrance- and exit-related speed-change lanes. The HSM methodology overpredicted all FI crashes, underpredicted multiple vehicle PDO crashes, and overpredicted single vehicle PDO crashes for entrance ramp segments. In the case of exit ramp segments, the HSM methodology underpredicted all multiple vehicle crashes and overpredicted all single vehicle crashes. The HSM methodology overpredicted all FI crashes and underpredicted all PDO crashes for both signal- and stop-controlled crossroad ramp terminals.

Cumulative residual plots and coefficient of variation were used to evaluate the quality of calibrated HSM-default SPFs. Results of calibration quality assessment indicated that estimated calibration factors were satisfactory for all freeway and ramp facilities considered in this study. However, for further accuracy and comparison purposes, calibration functions were developed to improve the fit to local data. Calibration functions were better fitted compared to calibrated HSM-default SPFs for freeway and ramp facilities in Kansas. Challenges faced, how those challenges were addressed, and data collection techniques used in this study are discussed. In summary, estimated calibration factors and developed calibration functions of this study would greatly improve making accurate decisions related to freeway and ramp safety in Kansas.

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Final Report

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PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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Abstract

Crash prediction models in the Highway Safety Manual (HSM) are used to quantify the safety experience of new and existing roadways. Safety Performance Functions (SPFs) or crash prediction models are statistical formulas developed on limited data from a few selected states, Kansas not being one of those states. Therefore, the HSM recommends calibration of HSM-default SPFs, or development of local SPFs, to enhance the accuracy of predicted crash frequency. This report demonstrates the HSM calibration procedure and its quality assessment for freeway segments, speed-change lanes, ramp segments, and crossroad ramp terminals in Kansas. The study used three years of recent crash data, the most recent geometric data, and HSM-recommended sample sizes for all facilities considered for the calibration.

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Cumulative residual plots and coefficient of variation were used to evaluate the quality of calibrated HSM-default SPFs. Results of calibration quality assessment indicated that estimated calibration factors were satisfactory for all freeway and ramp facilities considered in this study. However, for further accuracy and comparison purposes, calibration functions were developed to improve the fit to local data. Calibration functions were better fitted compared to calibrated HSM-default SPFs for freeway and ramp facilities in Kansas. Challenges faced, how those challenges were addressed, and data collection techniques used in this study are discussed. In summary, estimated calibration factors and developed calibration functions of this study would greatly improve making accurate decisions related to freeway and ramp safety in Kansas.

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Abbreviations

AASHTO	–	American Association of State Highway and Transportation Officials
NHTSA	–	National Highway Traffic Safety Administration
HSM	–	Highway Safety Manual
SHSP	–	Strategic Highway Safety Plan
HPMS	–	Highway Performance Monitoring System
SPF	–	Safety Performance Function
CMF	–	Crash Modification Factor
FHWA	–	Federal Highway Administration
KDOT	–	Kansas Department of Transportation
KCARS	–	Kansas Crash and Analysis Reporting System
CANSYS	–	Control Section Analysis System
EB	–	Empirical Bayes
NB	–	Negative Binomial
DVMT	–	Daily Vehicle Miles Travelled
AADT	–	Average Annual Daily Traffic
MV	–	Multiple Vehicle
SV	–	Single Vehicle
FI	–	Fatal and Injury
PDO	–	Property Damage Only
CURE	–	Cumulative Residual

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Chapter 1: Introduction

1.1 Background

Motor vehicle crashes are one of the top 10 causes of death in the United States (US). During 2017, 37,133 people were killed in roadway crashes in the US. This number was a decrease from 37,806 recorded in the previous year (NHTSA, 2019). However, there was a 6.5% increase in these type of deaths between 2015 and 2016; and an 8.4% increase in deaths between 2014 and 2015 in the US. The 8.4% increase was the second highest percentage increase recorded after the all-time record high of 9.4% between 1963 and 1964 (National Center for Statistics and Analysis, 2017). In Kansas, 461 people were killed due to roadway crashes in 2017—an increase from 429 deaths (+7%) recorded in 2016 (NHTSA, 2019). Nevertheless, this 7% increase in deaths between 2016 and 2017 was much lower than the 21% increase recorded between 2015 and 2016 (NHTSA, 2019). Table 1.1 tabulates the crash distribution on Kansas roadways by functional class from 2012 to 2016. For the period from 2012 to 2016, both fatal and injury (FI) crashes per mile and total crashes per mile were higher on interstate highways and other freeways/expressways than those for all other functional classes.

Table 1.1: Distribution of Crashes by Functional Class in Kansas (2012–2016)

Functional Class	Road Length in Miles (2016)	% Road Length	DVMT (2016)	% DVMT	FI Crashes (2012–2016)	FI Crashes per Mile	Total Crashes (2012–2016)	Total Crashes per Mile
Interstate	874	0.6	21,372,166	24.4	7,825	1.8	34,763	8.0
Other FWYS/EXPYS	595	0.4	8,915,631	10.2	3,798	1.3	16,116	5.4
Other Principal Arterial	2,946	2.1	12,355,244	14.1	18,490	1.3	76,108	5.2
Minor Arterial	5,630	4.0	18,729,175	21.4	15,679	0.6	64,769	2.3
Major Collector	24,210	17.0	13,275,745	15.1	8,023	0.1	36,669	0.3
Minor Collector	9,766	6.9	1,520,423	1.7	834	0	3,373	0.1
Local Road	98,026	69.0	11,543,471	13.2	13,756	0	69,274	0.1
All	142,047	100	87,711,855	100	68,405	0.1	301,072	0.4

Note: DVMT – Daily Vehicular Miles Traveled, FI – Fatal and Injury

Over the past years, safety-related decisions at the project level had been made based on engineering judgement and adherence to accepted national guidance. However, these tools do not allow comparison of safety performance of dissimilar roadway facilities or quantification of added safety benefits (Lubliner, 2011).

In 2010, the American Association of State Highway and Transportation Officials (AASHTO) published the first edition of the Highway Safety Manual (HSM) as a result of widespread road safety research conducted over the previous few decades (AASHTO, 2010). The project development process designed by AASHTO discusses the standard phases of a project, from planning to post-construction operations and maintenance (AASHTO, 2014). The roadway safety management process in Part B of the HSM plays a leading role in the project development process. The roadway safety management process is a cyclic procedure where network screening, diagnosis, countermeasure selection, project prioritization, and safety effectiveness evaluation take place within the cycle. Network screening is a major step in the roadway safety management process, where sites that require and benefit from safety treatment to improve highway safety performance are identified. Network screening using the Empirical Bayes (EB) method is the most preferred and reliable approach as it addresses the regression-to-mean (RTM) effect (Gan, Raihan, Alluri, Liu, & Saha, 2016). The RTM effect occurs when a site experiences high (low) crashes over a certain period, and it is statistically probable it will experience comparatively low (high) crashes in the following period of the same duration (AASHTO, 2010). The EB approach addresses the RTM effect by estimating the expected crash frequency of planning, design, operation, and maintenance decisions combining the predicted crash frequency with the observed crash frequency. The predicted crash frequency is estimated using a safety performance function (SPF), preferably calibrated to local conditions, which explains the relationship between exposure and mean crash frequency. SPFs for rural multilane highways, rural two-lane two-way roads, and urban and suburban arterials are provided in Part C of the HSM (AASHTO, 2010). Chapter 18 and Chapter 19 of the HSM, supplement to the first edition published in 2014, provides SPFs for freeway and ramp facilities. The calibration procedure for those facilities is provided in Appendix B of the HSM supplement (AASHTO, 2014).

1.2 Freeways in Kansas

The HSM defines freeways as fully access-controlled roadways having grade separation with all intersecting highways that can only be accessed through grade-separated interchanges (AASHTO, 2014). All interstate (I) highways and some United States (US) and Kansas (K) roadways (other freeways and expressways) in Kansas meet the HSM freeway criteria. Figure 1.1 highlights freeways in Kansas considered for this study. They are I-70, I-35, I-135, I-235, I-335, I-470, I-435, I-635, I-670, US-69, US-59, US-81, US-75, K-10, and K-96 roadways where the speed limit is more than or equal to 65 mph.

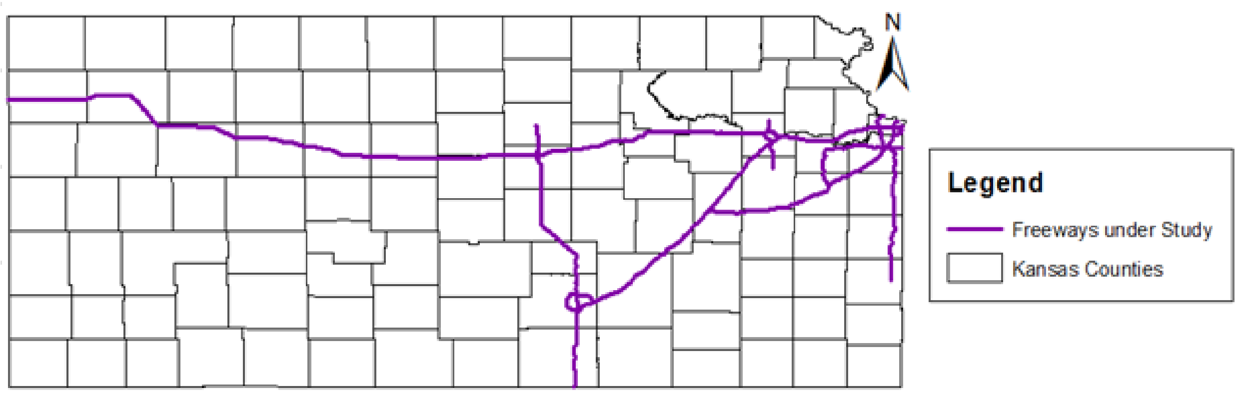


Figure 1.1: Freeways under Study

Freeway crash prediction models in the HSM are classified by area type (urban, rural) and cross section (4, 6, 8, and 10 through lanes) combinations, resulting in rural freeways with 4, 6, and 8 through lanes (R4F, R6F, and R8F) and urban freeways with 4, 6, 8, and 10 through lanes (U4F, U6F, U8F, and U10F). Table 1.2 provides mileage of freeways considered in this study by freeway types as defined in the HSM. In Kansas, a majority of freeways are rural 4-lane freeways (747 miles), whereas rural 8-lane and urban 10-lane freeways are not present within state boundaries.

Table 1.2: Mileage by Freeway Types in Kansas

Freeway Type	Freeway Mileage (miles)
Rural 4-lane (R4F)	746.99
Rural 6-lane (R6F)	11.23
Urban 4-lane (U4F)	207.36
Urban 6-lane (U6F)	54.00
Urban 8-lane (U8F)	7.37
Total	**1,025.93**

1.3 Problem Statement

According to the Kansas Strategic Highway Safety Plan (SHSP) published in 2017, the goal is to reduce fatalities by half for future years (KDOT, 2017b). Freeway crashes have a greater impact on motor vehicle fatalities in Kansas because freeways record the highest crash rates per mile, in both FI and total crashes, among all other roadway functional classes. It is, therefore, beneficial to have accurate crash prediction models for Kansas freeway and ramp facilities to make effective decisions in planning, design, operation, and maintenance processes, as KDOT maintains more than 1,000 miles of freeways.

SPFs provided in the HSM have been established for a set of base conditions distinct to each highway facility. These SPFs are statistical formulas developed from limited data gathered from a few selected states. For example, the original freeway crash prediction models in the HSM were developed based on data gathered from California, Maine, and Washington State (Bonneson, Geedipally, Pratt, & Lord, 2012). Weather conditions, animal population, topography, crash-reporting thresholds, highway conditions, driving culture, and lighting are some of the factors associated with crashes and are expected to be different from state to state. Therefore, use of HSM-default SPFs may result in biased crash frequency predictions compared to actual crash counts recorded each year. When applying HSM-default SPFs to a certain jurisdiction, calibration of these SPFs is highly recommended to avoid biased crash predictions. Jurisdictions may first calibrate existing HSM-default SPFs and assess the quality of the calibration factors before developing local SPFs, as HSM-default SPFs are already in place (Srinivasan, Carter, & Bauer, 2013). As the calibration factor is a single multiplier, the quality assessment of calibrated HSM-default SPFs is mainly conducted to capture inconsistencies of reported crashes among selected sites (Lyon, Persaud, & Gross, 2016). Development of local SPFs necessitates personnel with a high level of knowledge in statistical modeling; this is why transportation agencies without in-house expertise commonly hire university consultants (Srinivasan et al., 2013).

1.4 Study Objectives

Primary objectives of this research are as follows:

1. To apply the calibration procedure provided in Appendix B of the HSM to Kansas freeway segments, speed-change lanes, ramp segments, and crossroad ramp terminals.
2. To assess the quality of estimated calibration factors for all facility types considered in this study.
3. To estimate calibration functions when estimated calibration factors do not provide a better fit to local data.
4. To compare performance among estimated calibration factors; developed calibration functions; and estimated calibration factors by ranges of AADT and segment length using cumulative residual plots.
5. To provide recommendations for the best safety prediction approach for each facility type considered in this study.

1.5 Organization of the Report

This report consists of six chapters. Chapter 1 provides background on freeways in Kansas, freeway-related crashes, and the need for accurate crash prediction models. A review of previous research carried out on HSM calibrations, sample size guidelines, and SPF assessment are provided in Chapter 2. Chapter 3 explains required data variables, data collection techniques, and methodologies used in this study to develop calibration factors and calibration functions for freeway segments, speed-change lanes, ramp segments, and crossroad ramp terminals in Kansas. Estimated calibration factors; quality assessment of estimated calibration factors; developed calibration functions; and developed Kansas-specific crash proportions for freeway segments, speed-change lanes, ramp segments, and crossroad ramp terminals are provided in Chapter 4. Chapter 5 provides a research summary, conclusions, discussion of the application of calibrated crash prediction models, recommendations, and limitations based on research outcomes.

Chapter 2: Literature Review

The first part of this chapter is a discussion of calibration studies conducted in relation to freeway and ramp facilities, which are covered in Chapter 18 and Chapter 19 of the supplement to the first edition of the HSM (AASHTO, 2014). In the latter part of this chapter, past HSM calibration studies in Kansas, a review of sample size determination for calibration, and SPF assessment studies are discussed.

2.1 HSM Calibration for Freeways and Ramps

Shin, Lee, Dadvar, and Bharti (2016) carried out the most recent comprehensive freeway and ramp calibration study using Maryland data. The study used crash data from 2008 to 2010 to perform the calibration for freeway segments, speed-change lanes, and ramp terminals. However, this study did not estimate calibration factors for ramps and Collector-Distributor (C-D) roads due to insufficient crash data. The researchers obtained 60% of required geometric and condition data from the Maryland State Highway Administration database and the remaining data from Google Earth. Furthermore, the study mentioned that about 70% of the calibration effort was put into data gathering. Once required data were gathered, the study used the Interactive Highway Safety Design Model (IHSDM) developed by the Federal Highway Administration (FHWA) to compute local calibration factors (FHWA, 2019).

Results indicated that freeways, speed-change lanes, and ramp terminals in Maryland encountered fewer crashes compared to HSM base conditions during the study period. Table 2.1 summarizes estimated calibration factors for Maryland freeways, speed-change lanes, and ramp terminals. Other than the predictive models suggested in the HSM, the Maryland study also computed calibration factors separately for each area type (urban and rural), cross section type (4, 6, 8, and 10 lanes), crash type (multiple vehicle (MV) and single vehicle (SV)), severity type (fatal and injury (FI) and property damage only (PDO)), and control type (signal-controlled and stop-controlled) combinations. This was done because in HSM Chapter 18 and Chapter 19, HSM-default crash distributions or jurisdiction-specific crash distributions are applied after obtaining the calibration factors (AASHTO, 2014, pp.18-13, pp.19-15). Therefore, the Maryland study compared the sum of squared deviations (observed crashes minus predicted crashes) of calibrated facility types between HSM-default crash distributions and Maryland-specific crash distributions and provided recommendations accordingly. The study recommended Maryland-specific crash distributions for all freeway and ramp facilities considered in this study. However, disaggregated calibration factors were not recommended as no significant improvements were observed, and most of the facility types did not meet the HSM minimum sample size requirements.

Sun, Edara, Brown, Claros, and Nam (2013) estimated calibration factors for eight intersection types and five segment types provided in the first edition of the HSM using crash data from 2009 to 2011. This study also included calibration of three freeway types following the proposed freeway methodology in Appendix C of the HSM published in 2010. Data required for the calibration were gathered from a variety of sources, including the Transportation Management System (TMS) database of the Missouri Department of Transportation, Google street view photographs, and other sources. Samples were randomly selected for calibration considering geographic representativeness across the state. However, this sampling technique was not feasible for some freeway types as sites were located in only a few districts. Results for freeways showed calibration factors for FI crash models to be comparatively lower than PDO crash models. Table 2.2 tabulates estimated calibration factors for rural 4-lane, urban 4-lane, and urban 6-lane freeways in Missouri.

Table 2.1: Estimated Calibration Factors for HSM Chapter 18 and Chapter 19 in Maryland (2008–2010)

Facility Type	Crash & Severity Type	Number of Segments	Observed Crashes	Predicted Crashes	Calibration Factor
Freeways	MV FI	564	1,190	2,617.94	0.4546
	MV PDO	564	1,890	6,610.84	0.2859
	SV FI	564	910	1,451.53	0.6269
	SV PDO	564	1,735	2,705.70	0.6412
Speed-change Lanes	EN FI	264	358	605.63	0.5911
	EN PDO	264	600	1,139.64	0.5265
	EX FI	254	336	438.32	0.7666
	EX PDO	254	572	649.53	0.8806
Ramp Terminals	ST FI	147	83	122.85	0.6756
	SG FI	172	425	1,213.81	0.3501
	ST PDO	147	77	203.91	0.3776
	SG PDO	172	511	1,690.71	0.3022

Note: MV – Multiple Vehicle, SV – Single Vehicle, FI – Fatal and Injury, PDO – Property Damage Only, EN – Entrance-related, EX – Exit-related, ST – Stop-Controlled, SG – Signal-Controlled

Table 2.2: Estimated Freeway Calibration Factors in Missouri (2009–2011)

Facility Type	Crash & Severity Type	Number of Segments	Observed Crashes	Predicted Crashes	Calibration Factor
Rural 4-lane Freeways	MV FI	47	150	164.83	0.91
	MV PDO	47	645	325.76	1.98
	SV FI	47	268	348.05	0.77
	SV PDO	47	1,229	813.91	1.51
Urban 4-lane Freeways	MV FI	39	153	109.29	1.40
	MV PDO	39	669	186.35	3.59
	SV FI	39	142	202.86	0.70
	SV PDO	39	583	359.88	1.62
Urban 6-lane Freeways	MV FI	54	424	353.33	1.20
	MV PDO	54	1,482	909.20	1.63
	SV FI	54	206	203.96	1.01
	SV PDO	54	477	542.05	0.88

Note: MV – Multiple Vehicle, SV – Single Vehicle, FI – Fatal and Injury, PDO – Property Damage Only

Berry (2017) estimated calibration factors for urban 6-lane freeway segments in Missouri using crash data from 2012 to 2014. The sample of urban 6-lane segments used in this study was the exact same sample used by Sun et al. in 2013. However, the study followed methodology provided in Appendix B of the HSM supplement, where the high-volume parameter was introduced. Results showed the HSM predictive method underpredicted MV PDO crashes and overpredicted SV FI crashes, MV FI crashes, and SV PDO crashes for 6-lane freeways in Missouri. Table 2.3 shows estimated calibration factors for urban 6-lane freeways in Missouri.

Table 2.3: Estimated Freeway Calibration Factors in Missouri (2012–2014)

Facility Type	Crash & Severity Type	Number of Segments	Observed Crashes	Predicted Crashes	Calibration Factor
Urban 6-lane Freeways	MV FI	54	411	486	0.846
	MV PDO	54	1,281	1,050	1.220
	SV FI	54	189	196	0.964
	SV PDO	54	443	519	0.854

Note: MV – Multiple Vehicle, SV – Single Vehicle, FI – Fatal and Injury, PDO – Property Damage Only

Sun et al. (2016a) also estimated calibration factors for speed-change lanes, ramps, and ramp terminals in Missouri using crash data from 2010 to 2012. The study used the Enhanced Interchange Safety Analysis Tool (ISATe) for estimating calibration factors. One of the major challenges encountered in this study was the issue of locating crashes within the interchange area. Another study was conducted separately, where 12,409 crashes and 9,168 crash reports were manually reviewed to identify crashes at freeway interchange areas in Missouri (Sun et al., 2016b). All interchange facilities were manually classified according to HSM definitions, and sites were selected randomly maintaining geographical representativeness across the state. Whenever possible, the study managed to obtain 30 samples for the calibration by fulfilling the HSM minimum sample size requirement of 30 sites. Data required for the calibration were gathered from a variety of sources such as the TMS database, Google street-view photographs, web-based tools, and imported aerial photographs to AutoCAD. Table 2.4 provides estimated calibration factors for Missouri speed-change lanes, and Table 2.5 provides estimated calibration factors for Missouri ramps and ramp terminals. Sun et al. (2018) recalibrated 16 highway facilities in Missouri including rural 4-lane freeway segments, urban 4-lane freeway segments, and urban 6-lane freeway segments using crash data from 2012 to 2014. The study used the Interactive Highway Safety Design Model (IHSDM) software for the recalibration (FHWA, 2019).

Table 2.4: Estimated Calibration Factors for Missouri Speed-Change Lanes (2010–2012)

Facility Type	Crash & Severity Type	Number of Segments	Observed Crashes	Predicted Crashes	Calibration Factor
Rural Speed-change lanes	EN FI	30	3	4.201	0.714
	EN PDO	30	15	13.023	1.152
	EX FI	30	4	4.930	0.811
	EX PDO	30	13	11.184	1.162
Urban 4-lane Speed-change lanes	EN FI	30	6	10.026	0.598
	EN PDO	30	15	23.598	1.314
	EX FI	30	4	8.788	0.455
	EX PDO	30	11	21.192	0.519
Urban 6-lane Speed-change lanes	EN FI	30	20	46.375	0.431
	EN PDO	30	74	100.095	0.739
	EX FI	30	14	31.63	0.443
	EX PDO	30	40	82.991	0.482

Note: FI – Fatal and Injury, PDO – Property Damage Only, EN – Entrance-related, EX – Exit-related

Table 2.5: Estimated Calibration Factors for Missouri Ramp Facilities (2010–2012)

Facility Type	Crash & Severity Type	Number of Segments	Observed Crashes	Predicted Crashes	Calibration Factor
Rural Entrance Ramps	SV FI*	30	0	2.614	1.000
	SV PDO	30	3	3.900	0.769
	MV FI*	30	0	0.101	1.000
	MV PDO	30	1	0.402	2.489
Rural Exit Ramps	SV FI	30	2	5.611	0.356
	SV PDO	30	12	7.836	1.531
	MV FI*	30	0	0.027	1.000
	MV PDO*	30	0	0.163	1.000
Urban Entrance Ramps	SV FI	30	6	6.573	0.913
	SV PDO	30	12	10.704	0.121
	MV FI	30	3	1.119	2.681
	MV PDO	30	8	1.258	6.390
Urban Exit Ramps	SV FI	30	9	10.713	0.840
	SV PDO	30	20	15.798	1.266
	MV FI	30	2	0.850	2.354
	MV PDO	30	9	1.714	5.252
Rural D4	ST FI	32	7	8.302	0.843
	ST PDO	32	34	15.108	2.251
Urban D4	ST FI	30	23	18.765	1.226
	ST PDO	30	91	44.948	2.025
D4 with 2-lane crossroad	SG FI	30	84	77.300	1.087
	SG PDO	30	311	131.767	2.360
D4 with 4-lane crossroad	SG FI	32	161	188.842	0.853
	SG PDO	32	523	285.756	1.830
D4 with 6-lane crossroad	SG FI	10	88	100.744	0.874
	SG PDO	10	357	166.052	2.150
Rural A2	ST FI	16	2	6.905	0.290
	ST PDO	16	10	6.650	1.504
Urban A2	ST FI	23	19	18.359	1.035
	ST PDO	23	51	32.000	1.594
A2	SG FI	19	89	166.365	0.535
	SG PDO	19	273	232.924	1.172

Note: *denotes that the facility type did not have any crashes during the study period, a value of 1.000 (i.e. national data was used) MV – Multiple Vehicle, SV – Single Vehicle, FI – Fatal and Injury, PDO – Property Damage Only, EN – Entrance-related, EX – Exit-related, ST – Stop-Controlled, SG – Signal-Controlled

Srinivasan and Carter (2011) developed SPFs for rural and urban freeways within the influence of interchanges and outside the influence of interchanges in North Carolina. Smith, Carter, and Srinivasan (2017) developed calibration factors for rural 4-lane, urban 4-lane, urban 6-lane, and urban 8-lane freeways in North Carolina using crash data from 2009 to 2015 following the freeway models provided in NCHRP 17-45 (Bonneson et al., 2012). Lu, Haleem, Gan, and Alluri (2014) developed SPFs for Florida freeways and freeway interchange areas using the Negative Binomial (NB) method employing road geometry data from 2008 and reported crash data from 2007 to 2010. Then, the study compared the performance of calibrated SafetyAnalyst-default SPFs with Florida-specific SPFs. Results indicated Florida-specific SPFs were better fitted to local data than calibrated SafetyAnalyst-default SPFs. Michigan estimated calibration factors for all facility types covered in the first edition of the HSM in 2012 using data from 2005 to 2010 (MDOT, 2012). The study also included calibration of freeways and freeway interchange areas with regard to total crashes and FI crashes. Bonneson and Pratt (2008) developed calibration factors for rural freeways with four to eight lanes and urban freeways with four to ten lanes in Texas.

La Torre, Domenichini, Corsi, and Fanfani (2014) estimated calibration factors for freeway segments and speed-change lanes in Italy. The calibration was accomplished using 56 freeway sections having two, three, and four lanes. Crash data over a five-year period from 2005 to 2009 was considered for the analysis. Results showed a reasonable transferability of HSM predictive models to the Italian freeway network. It was identified that freeway segment models performed better compared to speed-change lane models because sample sizes for speed-change lanes were smaller. In addition, crash predictions for FI models were more accurate than PDO models.

La Torre, Meocci, Domenichini, Branzi, and Paliotto (2019) proposed a new methodology to transfer HSM freeway crash prediction models to European freeways. This study presented two crash prediction models for SV FI and MV FI crashes that could be applied on Italian rural freeways based on previously developed local SPFs. The dataset included FI crashes reported for a five-year period from 2009 to 2013 along 884 km of freeway segment length. In the interest of improving reliability of crash predictions, crash modification factors (CMFs) and calibration factors were developed and applied. Further, the goodness-of-fit of all models were compared using the Pearson's $\chi 2$ statistic, root mean square error, and residual analysis. Results presented a good fit of both models to the dataset used. Finally, these models were suggested for use as a tool for crash predictions on Italian freeways.

2.2 HSM Calibration Studies in Kansas

Dissanayake and Karmacharya (2020) estimated calibration factors and developed calibration functions for predicting crashes at urban intersections in Kansas. This study included four main urban intersection types: three-leg unsignalized intersections with stop control on the minor approach (3ST), three-leg signalized intersections (3SG), four-leg unsignalized intersections with stop control on the minor approach (4ST), and four-leg signalized intersections (4SG). The study period for 3ST, 3SG, and 4SG intersections was considered to be from 2013 to 2015, and for 4ST intersections the study period was taken between 2014 and 2016 as more recent data was available. Calibration factors showed the HSM methodology underpredicted both FI and total crashes for 4SG intersections and overpredicted both FI and total crashes for 3ST, 4ST, and 3SG intersections. This study compared the performance of calibration functions and calibration factors using two goodness-of-fit measures and concluded that calibration functions had better reliability as compared to calibration factors. Aziz and Dissanayake (2017) analyzed HSM calibration procedures for rural multilane segments and intersections in Kansas. Results showed the HSM methodology overpredicted FI crashes for 4-lane divided and 4-lane undivided segments and underpredicted total crashes for 4-lane divided and 4-lane undivided segments.

Lubliner (2011) conducted a study to analyze both the accuracy and practicality of using HSM prediction models for two-lane two-way highways in Kansas. However, due to limitations in the HSM calibration procedure, the study introduced an alternative calibration procedure focusing on prevalent animal crashes in Kansas. Bornheimer (2011) developed SPFs using the NB method for rural two-lane highways in Kansas. The study compared original HSM crash prediction models with Kansas-specific calibrated models and newly-developed crash prediction models to find the best model that fits rural two-lane highways in Kansas. Finally, the study recommended the two best models that would work for Kansas with and without animal crashes.

2.3 Site Selection for HSM Calibration

The HSM recommends a desirable sample size of 30 to 50 sites with at least 100 crashes per year for each predictive model calibration. For unbiased selection, sites need to be randomly selected without considering the number of crashes throughout the study period (AASHTO, 2014). Recent studies have identified that the HSM-suggested one-size-fits-all sample size is not adequate to obtain a reliable calibration factor. For example, collecting 100 crashes from 30 to 50 sites could be difficult in some instances (MnDOT, 2014; Xie, Gladhill, Dixon, & Monsere, 2011). A few studies mentioned that the use of a single criterion for calibration sample size might not be practical because each roadway type has a diverse set of characteristics and homogeneities, and further, data systems and the number of observed crashes vary considerably across states (Alluri, Saha, & Gan, 2016; Bahar, 2014; Banihashemi, 2012; Kim, Anderson, & Gholston, 2015; Trieu, Park, & McFadden, 2014).

Since recent studies have stipulated the one-size-fits-all sample size suggested by the HSM may not provide anticipated results accurately, the study explored sampling techniques previously used in other HSM calibration studies. Banihashemi (2012) estimated calibration factors using an entire data set (population) for different roadway types, and these factors were considered as ideal calibration factors. Then, for different subsets, the sensitivity of calibration factors were estimated such that the probability of generating calibration factors for each subset falls within 5% and 10% of ideal calibration factors. The study concluded that some types of roadways required a large sample-size number for an effective calibration. Alluri et al. (2016) followed the methodology suggested by Banihashemi (2012) and concluded that calibration factors obtained from reliable sample sizes are more likely to lie within 10% of ideal calibration factors. Shirazi, Lord, and Geedipally (2016) studied the required sample size for HSM calibrations based on the coefficient of variation (CV), which is the ratio of the standard deviation to the mean observed crashes. Shin, Yu, Dadvar, and Lee (2015) proposed a method to determine minimum sample size based on the finite population correction (FPC) method allowing for adjustments between desired error levels of the estimated calibration factors, confidence levels, and sample standard deviations. Trieu et al. (2014) conducted a sensitivity analysis to evaluate accuracy of the HSM-recommended sample size. Samples were obtained from the entire data set at various percentages and the Monte Carlo simulation was performed with an initial iteration such as 500. It was found that when sample size increased, calibration factors with higher errors decreased. To sum up, the FPC method proposed by Shin et al. (2015)

seemed to be a common choice in previous HSM calibration studies for determining an appropriate sample size.

2.4 Assessing the Quality of the Calibration Process

Srinivasan, Colety, Bahar, Crowther, and Farmen (2016) assessed the goodness-of-fit of rural two-lane roadway SPFs in Arizona using cumulative residual (CURE) plots. Even though the estimated calibration factor was much closer to 1.000 for total crashes, when the CURE plot was created for fitted values, most of the cumulative residuals were lying outside the confidence limits, indicating a poor goodness-of-fit. The study also estimated calibration factors by exposure variables such as AADT, segment length, and alignment. In order to improve the goodness-of-fit, the study explored different forms of calibration functions using the NB method, Poisson regression, and Ordinary Least Squares Method. The study showed the base calibration function (i.e., the relationship between predicted crashes and observed crashes) and calibration functions developed using AADT and segment length fitted better to the data set than other forms of calibration function.

Claros, Sun, and Edara (2018) performed a comparative analysis among four safety prediction techniques: HSM calibration factors, calibration factors by ranges of exposure variable, calibration function, and local SPFs using 160 urban 4-lane freeway segments in Missouri. The study mentioned that in general, prediction accuracy increases from the calibration factor to local SPFs as shown in Figure 2.1. The NB method was used to develop calibration functions and CURE plots were used to compare the goodness-of-fit among prediction techniques. Log-likelihood and inverse overdispersion were assessed for calibration functions and local SPFs in addition to CURE plots. Calibration factors by AADT ranges had the highest goodness-of-fit among those compared to HSM calibration factors. Calibration functions did not show a significant improvement in accuracy compared to calibration factors by ranges of exposure. Local SPFs presented a similar goodness-of-fit as the calibration factor and calibration function by AADT ranges.

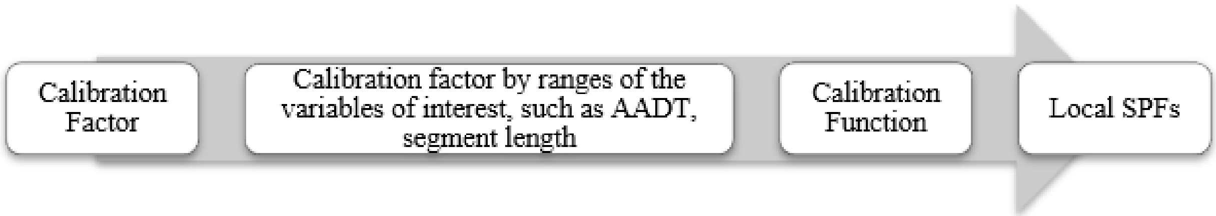


Figure 2.1: Accuracy of Safety Prediction Techniques
Claros et al. (2018)

Vargas, Raihan, Alluri, and Gan (2019) compared the performance of Florida-specific SPFs with calibrated SafetyAnalyst-default SPFs using several goodness-of-fit measures such as mean absolute deviation, mean squared predicted error, and Freeman-Tukey R-square in predicting crashes on rural and urban two-lane and multi-lane highway facilities in Florida. This study was conducted to identify the need for developing Florida-specific SPFs for these facilities. Results showed Florida-specific SPFs generally produced better-fitted models compared to calibrated SafetyAnalyst-default SPFs. However, calibrated SafetyAnalyst-default SPFs performed better compared to existing Florida-specific SPFs when calibrated to the latest crash data.

In summary, the literature shows performance of calibrated HSM-default SPFs, local jurisdiction-specific SPFs, and calibration functions are highly dependent on the input data set unique to each jurisdiction. For example, calibration functions seem to fit better to local data for some jurisdictions than calibration factors developed for a range of exposure variables. Therefore, jurisdictions must discover which safety prediction techniques provide a better fit for local data at each facility type.

Chapter 3: Data and Methodology

This chapter outlines the HSM calibration procedure for freeway and ramp facilities along with offering an overview of data preparation. The calibration procedure requires two main types of data from the selected sites: (1) geometric and traffic data and (2) crash data.

3.1 Definition of Freeway Segment and Speed-Change Lane

A *freeway segment* is defined as a length of roadway consisting of 'n' number of through lanes with a continuous cross section providing two directions of travel, where travel lanes are physically separated by either distance or a barrier. A *speed-change lane* is defined as a section of roadway located between the gore and taper points of a ramp's merge and diverge area as shown in Figure 3.1 (AASHTO, 2014). There are two types of speed-change lanes: ramp entrance-related speed-change lanes (SCen) and ramp exit-related speed-change lanes (SCex). According to the HSM, all crashes occurring in the shaded area "A" in Figure 3.1 should be classified as speed-change lane crashes and all crashes occurring in the region "B" should be classified as freeway segment crashes.

Figure 3.2 schematically shows three shaded freeway segments (Fs1, Fs2, and Fs3) and two speed-change lanes (SCen and SCex). In cases where a speed-change lane is presented within a freeway segment, calculation of the effective freeway segment length is required. In this study, freeway segments connected to ramps were excluded from consideration because those segments were used to create the speed-change lane database. The detailed segment reduction procedure conducted in creating the freeway segment database and speed-change lane database used in the research is explained in Section 3.7 of this report.

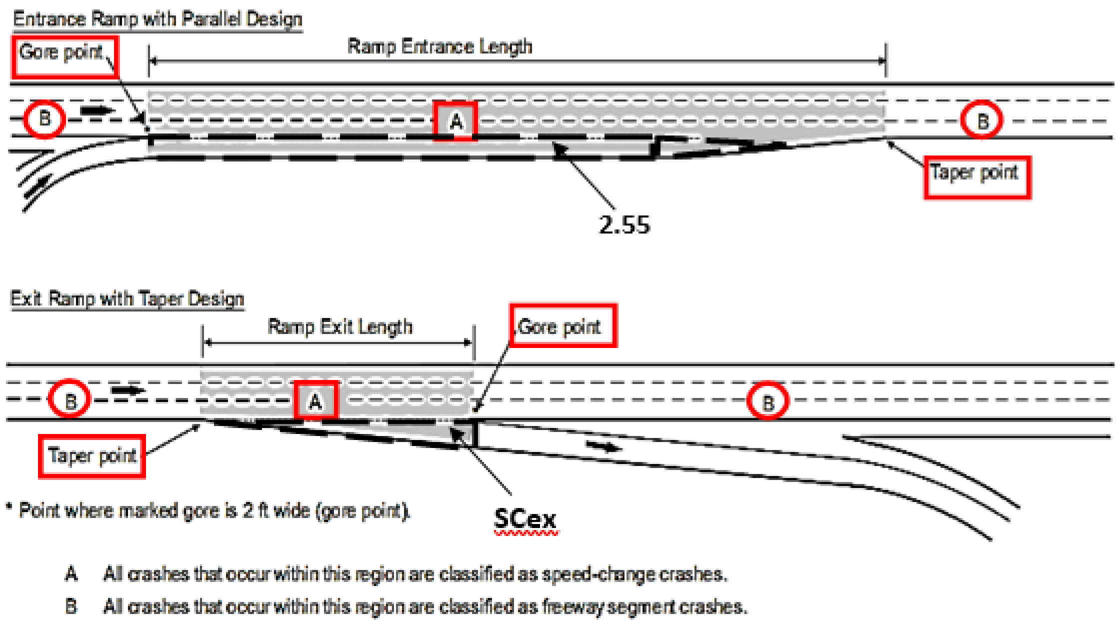


Figure 3.1: Assigning Crashes to Freeway Segments and Speed-Change Lanes

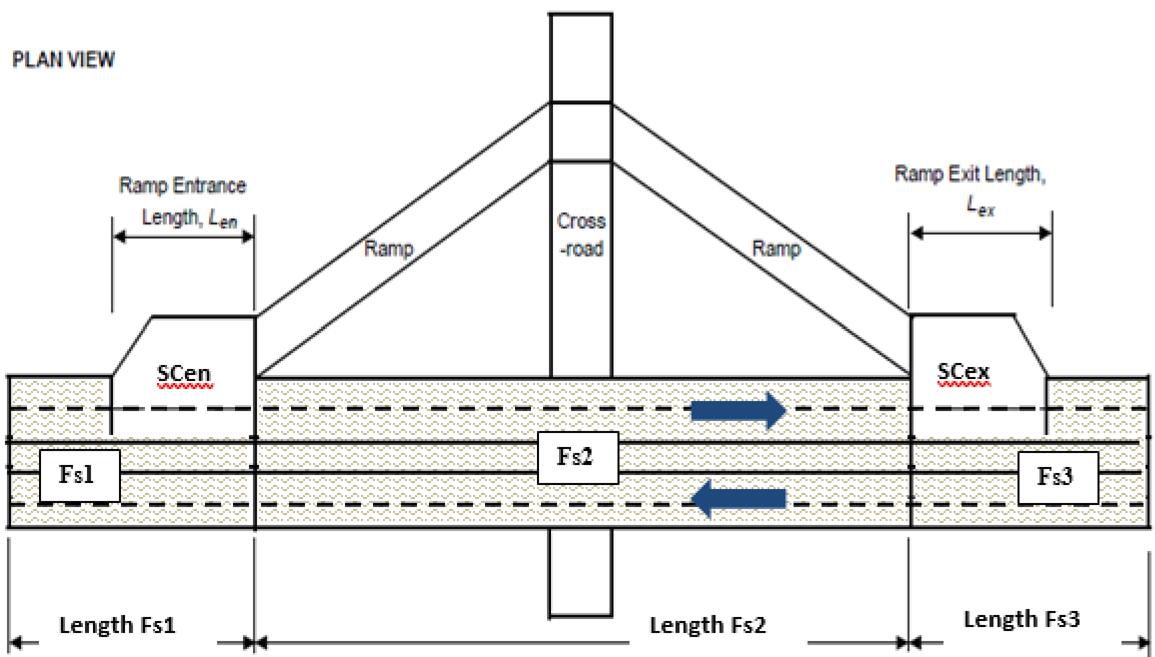


Figure 3.2: Illustration of Freeway Segments and Speed-Change Lanes
AASHTO (2014)

3.2 Definition of Ramp Segment, C-D Road, and Crossroad Ramp Terminal

A *ramp segment* and *Collector-Distributor* (C-D) road segment is defined as a length of roadway consisting of 'n' number of through lane(s) with a continuous cross section providing one direction of travel. A *crossroad ramp terminal* is a controlled terminal between a ramp and crossroad (AASHTO, 2014). According to the HSM, any crashes that occur on a ramp or C-D road are classified as either intersection-related or segment-related crashes (AASHTO, 2014, pp. 19-27). Intersection-related crashes are assigned to the corresponding crossroad ramp terminal, which has an influence area that extends 250 ft in each direction along the crossroad and ramps as shown in a dotted box in Figure 3.3 (AASHTO, 2014, pp. 19-26). Segment-related crashes are assigned to the corresponding ramp or C-D segment as shown in dashed boxes in Figure 3.3, which schematically illustrates two entrance ramp segments (Ren1 and Ren2), two exit ramp segments (Rex1 and Rex2), five C-D road segments (CD1, CD2, CD3, CD4, and CD5), and one crossroad ramp terminal (In). The three control types for crossroad ramp terminals addressed in the HSM are signal-controlled, all-way stop-controlled, and one-way stop-controlled. C-D roads are more likely to be an urban design with a main purpose of moving vehicle lane changing away from high-speed traffic on freeway main lanes (Texas A&M Transportation Institute, n.d.). In Kansas, more than 90% of the 141,173 centerline miles of roadways are rural roads. Therefore, the calibration for C-D roads was not conducted due to a lack of sites that match the HSM definition.

3.2.1 Crossroad Ramp Terminal Configurations in Kansas

The seven most common types of crossroad ramp terminal configurations are addressed in the HSM labeled as D3en, D3ex, D4, A4, B4, A2, and B2 as shown in Appendix A. When applying the HSM predictive method to crossroad ramp terminals, the procedure is specific to each configuration (AASHTO, 2014). Table 3.1 provides a summary of crossroad ramp terminal configurations present on freeways considered in this study. As defined in the HSM, D3en, D3ex, A2, and B2 are three-leg terminals, and D4, A4, and B4 are four-leg terminals. In Kansas, the majority of crossroad ramp terminals are the "D4" type used at diamond interchanges with diagonal ramps. Other than the crossroad ramp terminal configurations addressed in the HSM, 11 roundabout ramp terminals and three unique types of ramp terminals present on freeways are considered in this study as shown in Appendix B.

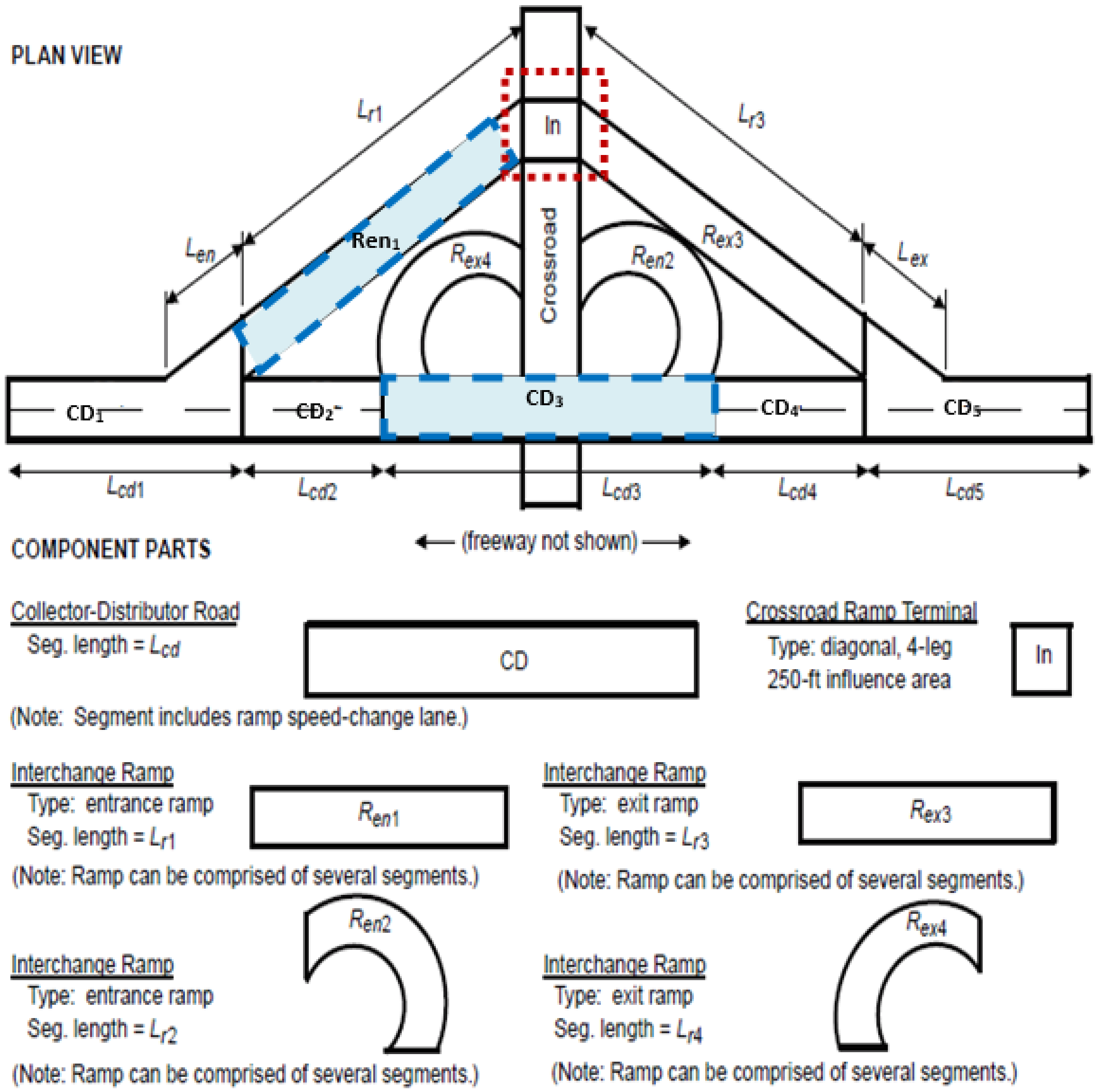


Figure 3.3: Illustration of Ramp Segments, C-D Roads, and Crossroad Ramp Terminals
AASHTO (2014)

Table 3.1: Crossroad Ramp Terminal Configurations of Freeways under Study

Freeway	D3en	D3ex	D4	A4	B4	A2	B2	Roundabout	Unique
I-35	0	0	73	0	1	12	4	2	0
I-70	6	8	168	0	0	17	15	6	1
I-135	5	5	62	3	0	5	4	2	0
I-235	1	1	13	0	0	3	3	0	1
I-335	0	0	1	0	0	1	0	0	0
I-435	0	1	20	2	1	4	5	0	1
I-470	4	4	7	0	0	1	0	0	0
I-635	3	3	5	2	0	3	2	0	0
US-59	1	1	10	0	0	0	0	0	0
US-69	1	1	45	3	0	1	1	0	0
US-75	0	0	13	0	0	1	0	1	0
US-81	0	0	6	0	0	0	0	0	0
K-10	0	0	16	0	0	3	3	0	0
K-96	3	3	16	0	0	2	2	0	0
Total	**24**	**27**	**455**	**10**	**2**	**53**	**39**	**11**	**3**

3.3 Data Preparation

3.3.1 Main Data Sources

KDOT maintains two separate databases for geometric/traffic attributes and reported crashes on Kansas roadways. The Control Section Analysis System (CANSYS) database is comprised of data on roadway classifications, geometrics, and the condition of more than 10,000 miles of state roadways in Kansas (KDOT, 2011). The Kansas Crash Analysis and Reporting System (KCARS) database is comprised of crash records reported by police in Kansas. In addition, KDOT also maintains yearly AADTs of Kansas roadways in ArcGIS shapefiles, which are submitted to the Highway Performance Monitoring System (HPMS) each year (FHWA, 2016).

3.3.2 Study Period

The HSM recommends the calibration period to be a duration that is a multiple of 12 months to avoid seasonal effects (AASHTO, 2014). Shorter periods are highly unpredictable due to the randomness of crashes, and longer periods are subjected to alterations in reporting thresholds or physical changes in highway elements (Lubliner, 2011). A three-year period from 2013 to 2015 was considered as the study period for the freeway segment and speed-change lane calibration, which was the period with most recent data available at the beginning of this study. The study period for the ramp segment and crossroad ramp terminal calibration was set from 2014 to 2016 due to the availability of more recent data as research progressed.

3.4 Data Required for HSM Calibrations

3.4.1 Freeway Segments and Speed-Change Lanes

The HSM does not provide in-depth guidelines for collecting data needed for the calibration, as data systems are different for each state (Brown, Sun, & Edara, 2014). Table 3.2 provides data needed for the freeway segment and speed-change lane calibration, including sources of data extraction. Even though the majority of geometric data elements were readily available in the CANSYS database, most of those required manual adjustments to conform to HSM definitions. A few data elements were collected using Google Earth and ArcGIS tools, as they were not available in the CANSYS database (Esri, 2012). Accordingly, 16 and 13 data elements were needed to perform the freeway segment and speed-change lane calibration, respectively. Few data elements were desirable for both facility types; however, these desirable variables, such as clear zone width, length of rumble strips on inside/outside shoulders, and proportion of AADT that occurs during hours where lane volume exceeds 1,000 veh/hr/ln were sensitive to relevant CMFs. As a result, data for all elements in Table 3.2 were collected from a variety of data sources.

3.4.2 Ramp Segments and Crossroad Ramp Terminals

Table 3.3 provides data needed for the ramp segment calibration including sources of data extraction. More than 90% of the geometric data elements needed for the ramp segment calibration were collected using Google Earth.

Table 3.2: Sources of Data in Calibrating Freeway Segments and Speed-Change Lanes

Data Element	R/D*	Freeway Segments	Speed-Change Lanes	Source
Area type (Urban & Rural)	R	√	√	CANSYS
Number of through lanes	R	√	√	CANSYS
Segment length	R	√	√	CANSYS
Length of radii and horizontal curves	R	√	√	CANSYS
Lane width	R	√	√	CANSYS
Paved inside/outside shoulder width	R	√	√	CANSYS
Median width	R	√	√	CANSYS
Length of rumble strips on inside/outside shoulders	D	√	√	CANSYS
AADT volume of freeway	R	√		CANSYS
AADT volume of ramp in Speed-change lane	R		√	HPMS GIS shapefiles
Length of and offset to median barrier	R	√	√	CANSYS/Google Earth
Length of and offset to outside barrier	R	√		Google Earth
Clear zone width	D	√		Google Earth
AADT volume of and distance to nearest upstream entrance ramp	R	√		HPMS GIS shapefiles/GIS Measure Tool
AADT volume of and distance to nearest downstream exit ramp	R	√		HPMS GIS shapefiles/ GIS Measure Tool
AADT volume of freeway adjacent to Speed-change lane	R		√	CANSYS
Proportion of AADT that occurs during hours where lane volume exceeds 1,000 veh/hr/ln	D	√	√	Calculated using the formula provided in section 18.4.2 of the HSM
Presence and length of Type B weaving sections	R	√	√	No Type B weaving sections present in selected segments

Note*: R – Required, D – Desired

Table 3.3: Sources of Data in Calibrating Ramp Segments

Data Element	R/D*	Ramps	Source
Area type (Urban & Rural)	R	√	CANSYS
Number of through lanes	R	√	Google Earth
Segment length	R	√	GIS shapefiles
AADT volume of the ramp	R	√	GIS shapefiles
Length of radii and horizontal curves	R	√	KDOT curve tool/ GIS shapefiles
Lane width	R	√	Google Earth
Paved left/right shoulder width	R	√	Google Earth
Length of and offset to left side barrier	R	√	Google Earth
Length of and offset to right side barrier	R	√	Google Earth
Presence of lane add or drop	D	√	Google Earth
Presence of Speed-change lane	R	√	Google Earth
Presence and length of weaving section	R		Google Earth

Note*: R – Required, D – Desired

Table 3.4 provides data needed for the calibration of stop-controlled and signal-controlled crossroad ramp terminals including sources of data extraction. According to the HSM, 15 variables were needed to perform the stop-controlled crossroad ramp terminal calibration and 20 variables were needed to perform the signal-controlled crossroad ramp terminal calibration. As with ramp segments, more than 90% of geometric data required for the stop-controlled and signal-controlled crossroad ramp terminal calibration were collected using Google Earth. Data variables needed for both ramp segment and crossroad ramp terminal calibration were extremely challenging to gather compared to freeway facility calibration data variables. Specifically, data gathering for the calibration of ramp segments and crossroad ramp terminals was the most time-consuming and challenging task of this entire study.

Table 3.4: Sources of Data in Calibrating Crossroad Ramp Terminals

Data Element	R/D*	One-Way Stop-Controlled Crossroad Ramp Terminals	Signal-Controlled Crossroad Ramp Terminals	Source
Area type (Urban & Rural)	R	√	√	CANSYS
Ramp Terminal Configuration	R	√	√	Google Earth
Type of Traffic Control	R	√	√	Google Earth
Control for exit ramp right turn movement	R	√	√	Google Earth
AADT for the inside and outside crossroad legs	R	√	√	GIS shapefiles
AADT volume for each ramp leg	R	√	√	GIS shapefiles
Number of through lanes on each crossroad approach	R	√	√	Google Earth
Number of Lanes on the exit ramp	R	√	√	Google Earth
Number of crossroad approaches with left turn lanes	R	√	√	Google Earth
Number of crossroad approaches with right turn lanes	R	√	√	Google Earth
Number of unsignalized public street approaches to the crossroad leg outside of the interchange	D	√	√	Google Earth
Distance to next public street intersection	D	√	√	Google Earth
Distance to adjacent crossroad ramp terminal	D	√	√	Google Earth
Crossroad median width and left turn width	R	√	√	Google Earth
Skew Angle	R	√		Google Earth
Number of unsignalized driveways on the crossroad leg outside of the interchange	D		√	Google Earth
Number of crossroad approaches with protected-only left-turn operation	R		√	Google Earth
Number of crossroad approaches with right-turn channelization	R		√	Google Earth
Presence of exit ramp right-turn channelization	R		√	Google Earth
Presence of non-ramp public street leg	D		√	Google Earth

Note*: R – Required, D – Desired

3.5 Extraction of Geometric Data

3.5.1 Freeway Segment and Speed-Change Lane Calibration

As emphasized in Section 3.4, approximately 65% of geometric and traffic data needed to perform freeway segment and speed-change lane calibrations were obtained from the CANSYS database. In addition to roadway geometric and condition data, CANSYS also includes data on at-grade rail crossings, bridges, and access permits (KDOT, 2011). The geometric and accident data unit (GAD) in KDOT maintains CANSYS, and the data are collected by numerous entities within KDOT. Geometric data elements needed to perform the freeway segment and speed-change lane calibration are illustrated in this section.

3.5.1.1 Access Control

HSM defines freeways as fully access-controlled roadways having grade separation with all intersecting roadways that can only be accessed through grade-separated interchanges (AASHTO, 2014). The access control in CANSYS is coded at three levels: ‘01’ = full access control, ‘02’ = partial access control, and ‘03’ = no access control of conditions. For this study, all fully access-controlled roadways were selected first, and a segment reduction procedure was carried out later with regard to HSM definitions.

3.5.1.2 Area Type

Depending on roadway characteristics, surrounding land uses, and population, the HSM classifies an area as urban, suburban, or rural (AASHTO, 2014). The definition for area type in the HSM is based on FHWA guidelines, which define “urban” areas as localities where the population is greater 5,000, and “rural” areas as localities where the population is less than 5,000 (AASHTO, 2014). In addition, the HSM refers to “suburban” areas as the outlying portions of an urban area. KDOT practices the same area classification definitions and the CANSYS database includes a field named “RURAL_URBAN” that indicates the area type corresponding to each segment.

3.5.1.3 Number of Through Lanes

The total number of through lanes for freeway segments are counted by considering both directions of travel together. For speed-change lanes, the number of through lanes are counted in the portion of freeway adjacent to the speed-change lane, including those freeway lanes in the opposing travel direction (AASHTO, 2014). Figure 3.4 provides an example of how the number of through lanes are counted in cases where a lane adds or a lane drops. According to guidelines provided in the HSM, high occupancy lanes and managed lanes should be excluded from the through lane count; and auxiliary lanes should be included in the through lane count if the weaving length exceeds 0.85 miles. Speed-change lanes that merge with or diverge from the freeway should be included in the through lane count if the length exceeds 0.3 miles (AASHTO, 2014).

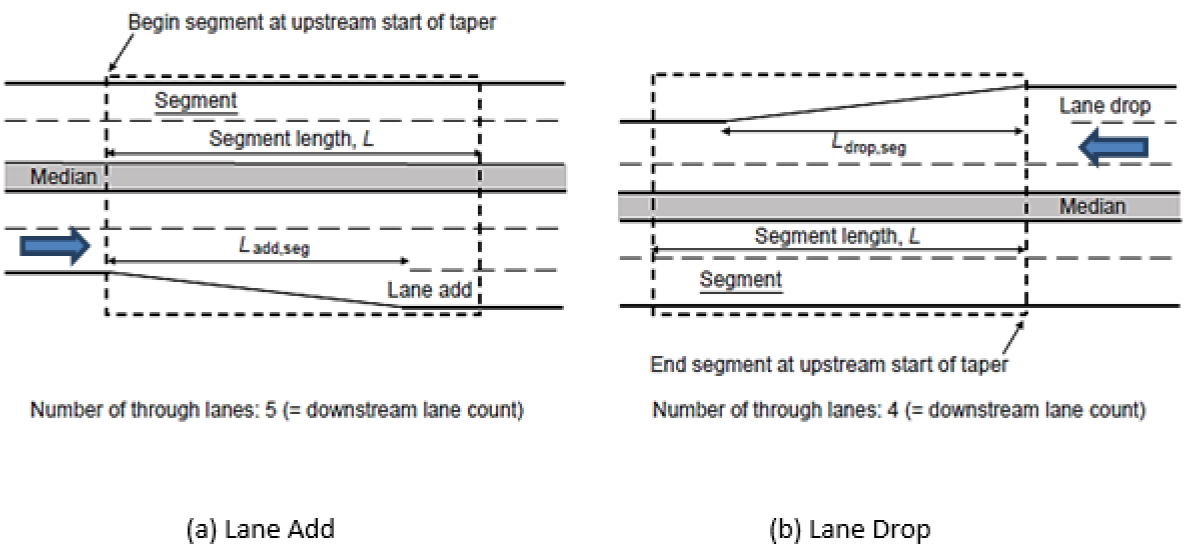


Figure 3.4: Through Lane Count in Segments in Cases Where a Lane Adds or Drops
AASHTO (2014)

3.5.1.4 Segment Length

Segment length is the distance from the beginning to the end of a segment. A segment may include different components, such as speed-change lanes, lane add/drop, and auxiliary lanes, but only if they meet the criteria mentioned in Section 3.5.1.3. The HSM recommends a segment be between 0.1 and 1 mile in length for carrying out of a freeway segment calibration (AASHTO, 2014). Segments shorter than 0.1 miles were excluded from the study and segments longer than 1 mile were divided into 1-mile sections using the ET Geo Wizards Tool (Version 11.3; ET Spatial Techniques, 2019). The same tool was used in the Maryland freeway calibration study to divide freeway segments into 1-mile sections (Shin et al., 2016). As an example, Figure 3.5 shows 1-mile sections created from the ET Geo Wizards tool in a 2.55 mile long segment.

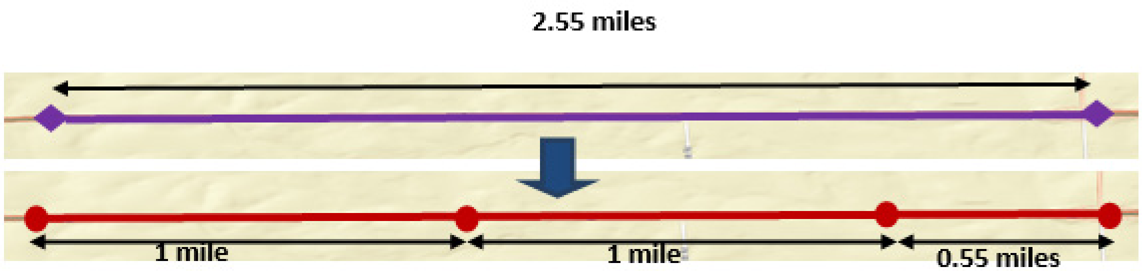


Figure 3.5: One-mile Sections Created by ET Geo Wizards Tool in a 2.55 Mile Long Freeway Segment

According to the HSM, the length of a speed-change lane should be limited to 0.3 miles and if it exceeds 0.3 miles, then the speed-change lane is counted as a through lane (AASHTO, 2014, pp. 18-15). However, the HSM does not provide guidance on the minimum length to be considered for entrance or exit speed-change lanes. For this research, the minimum length for entrance-related speed-change lanes was considered to be 0.04 miles, and minimum length for exit-related speed-change lanes was considered to be 0.02 miles based on applicability of ramp-entrance and ramp-exit CMFs provided in the HSM. However, Shin et al. (2016) used 0.05 miles as the minimum length for both entrance and exit speed-change lanes in the Maryland study.

3.5.1.5 Begin/End Mile Post

In Kansas, following US customs, mileposts increase from South to North for odd numbered routes and West to East for even-numbered routes. KDOT uses two milepost systems, namely, a state milepost system and a county milepost system. State mileposts begin from the southern or western state lines, whereas county mileposts begin likewise from the county lines.

Data used in this study were collected using the state milepost system. Therefore, the difference between beginning and ending mileposts was the segment length.

3.5.1.6 Effective Segment Length

As mentioned in Section 3.1, calculation of effective length (L*) is required when a speed-change lane is present in a freeway segment. Figure 3.6 illustrates the typical calculation of the effective segment length (L*), which is the segment length minus the length of speed-change lanes, in miles. However, for this research, calculation of effective segment length was not required because the study selected only basic freeway segments that are not connected to ramps for the freeway segment calibration. As the CANSYS database did not create a new homogenous segment with a ramp being present, the study had to use freeway segments connected to ramps to create the speed-change lane database.

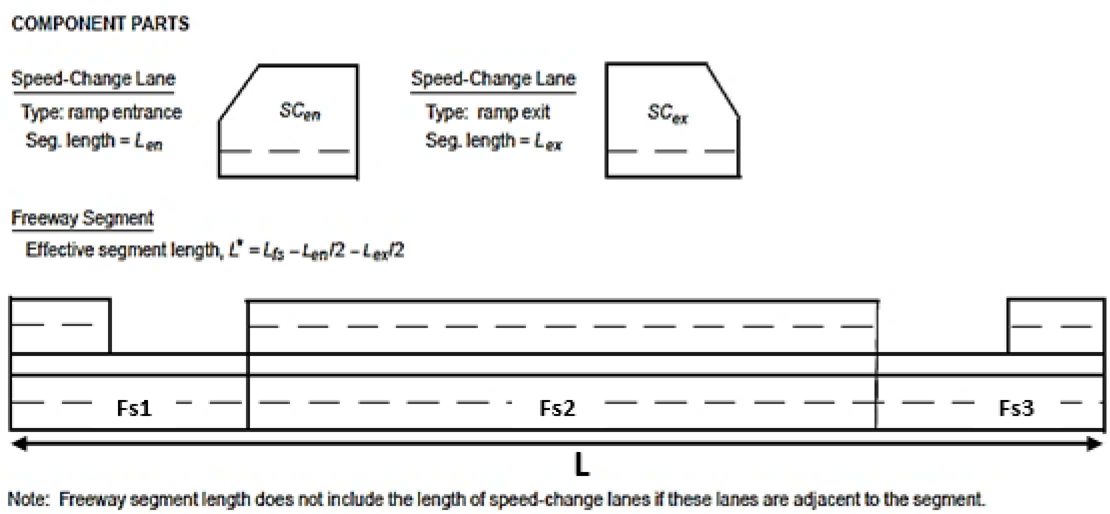


Figure 3.6: Calculation of Effective Segment Length
AASHTO (2014)

3.5.1.7 Lane Width and Paved Inside/Outside Shoulder Width

Lane width is measured at successive points along the roadway and averaged for all through lanes as per HSM guidelines. If required, average lane width is rounded to the nearest 0.5 ft. Paved inside/outside shoulder width is also measured at successive points along the roadway (AASHTO, 2014). If required, paved inside/outside shoulder width is rounded to the nearest 1 ft.

3.5.1.8 Median Width

Median width is the distance between the inside (left) edges of the traveled way for two roadways in both travel directions including the paved inside shoulder width (if present). This measurement is estimated as an average of the median widths at different points of the segment. If necessary, the average median width is rounded to the nearest 10 ft (AASHTO, 2014). If the average median width exceeds 90 ft, it is to be set to 90 ft.

3.5.1.9 Radius of Curve and Length of Curve

If a curve is present in a segment, the radius of curve, length of curve (curve length), and length of curve in the segment need to be calculated to perform the freeway facility calibration. The radius of curve is measured in feet for each roadbed separately along the inside edge of the traveled way (AASHTO, 2014). In case of a spiral curve, the radius of the central circular portion of the curve should be considered. If the curve is present in both directions of the roadway (in both roadbeds), the equivalent curve radius is to be calculated using Equation 3.1. If the curve is present only in one direction (in one roadbed), only $R^2_{a,i}$ should be used.

$$\boldsymbol{R_i^*} = \left(\frac{\mathbf{0.5}}{\boldsymbol{R_{a,i}^2}} + \frac{\mathbf{0.5}}{\boldsymbol{R_{b,i}^2}}\right)^{\mathbf{-0.5}} \qquad \textbf{Equation 3.1}$$

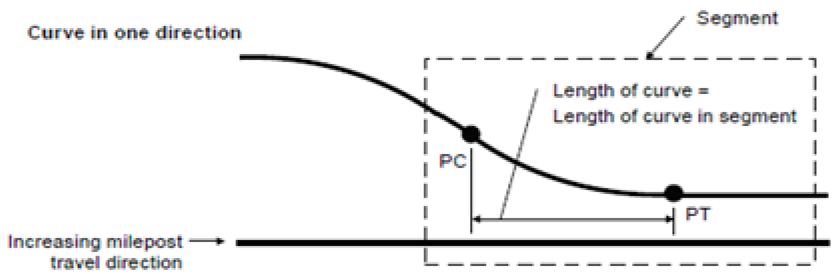
Where:

R_i^* = Equivalent radius of curve *i*,

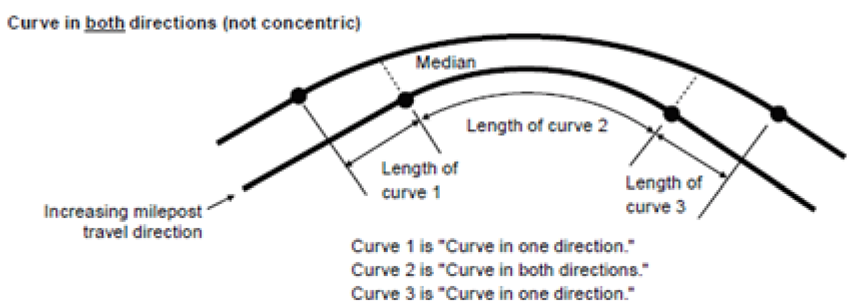
$R_{a,i}^2$= Radius of curve *i* in one roadbed, and

$R_{b,i}^2$= Radius of curve *i* in second roadbed.

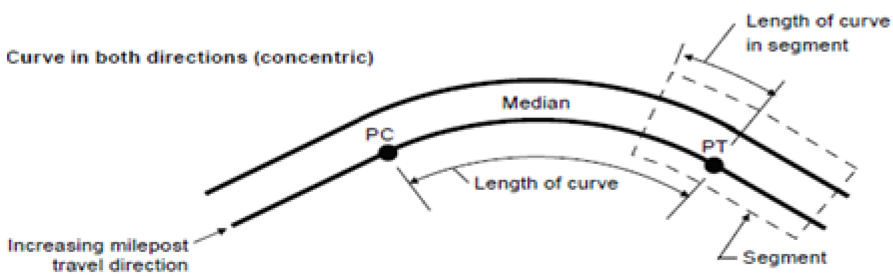
The length of curve is measured along the reference line from the point where the tangent ends and the point of curvature (PC) begins to the point where the curve ends and the point of tangent (PT) begins. The length of curve in the segment is measured within the boundaries of the desired freeway segment or speed-change lane (AASHTO, 2014). In addition, this measurement should not exceed the segment length or curve length. Figure 3.7 illustrates the length of curve and length of curve in the segment for different scenarios of curves that could be present on a freeway facility. Specifically, Figure 3.7(a) represents a curve present in one direction of the segment, Figure 3.7(b) signifies a not-concentric curve present in both directions of the segment, and Figure 3.7(c) signifies a concentric curve present in both directions of the segment.



(a) Curve in one direction (in one roadbed)



(b) Not-concentric curve in both directions (in both roadbeds)



(c) Concentric curve in both directions (in both roadbeds)

Figure 3.7: Curve Radius and Curve Length in a Segment

AASHTO (2014)

3.5.1.10 Length of Rumble Strips and Length of Median/Outside Barrier

Length of rumble strips is measured separately for each shoulder type and travel direction. Length of the median/outside barrier is measured along the reference line for each short piece of barrier and travel direction.

3.5.1.11 Length and Offset to Median/Outside Barrier

Barrier offset is the distance from the nearest edge of the traveled way including the inside shoulder to the barrier face. Barrier length is the length of lane paralleled by a barrier; it is a total for both travel directions (AASHTO, 2014). As the CANSYS database did not include measurements for length of and offset to the median/outside barrier, this study estimated those variables using the ruler tool in Google Earth as shown in Figure 3.8.

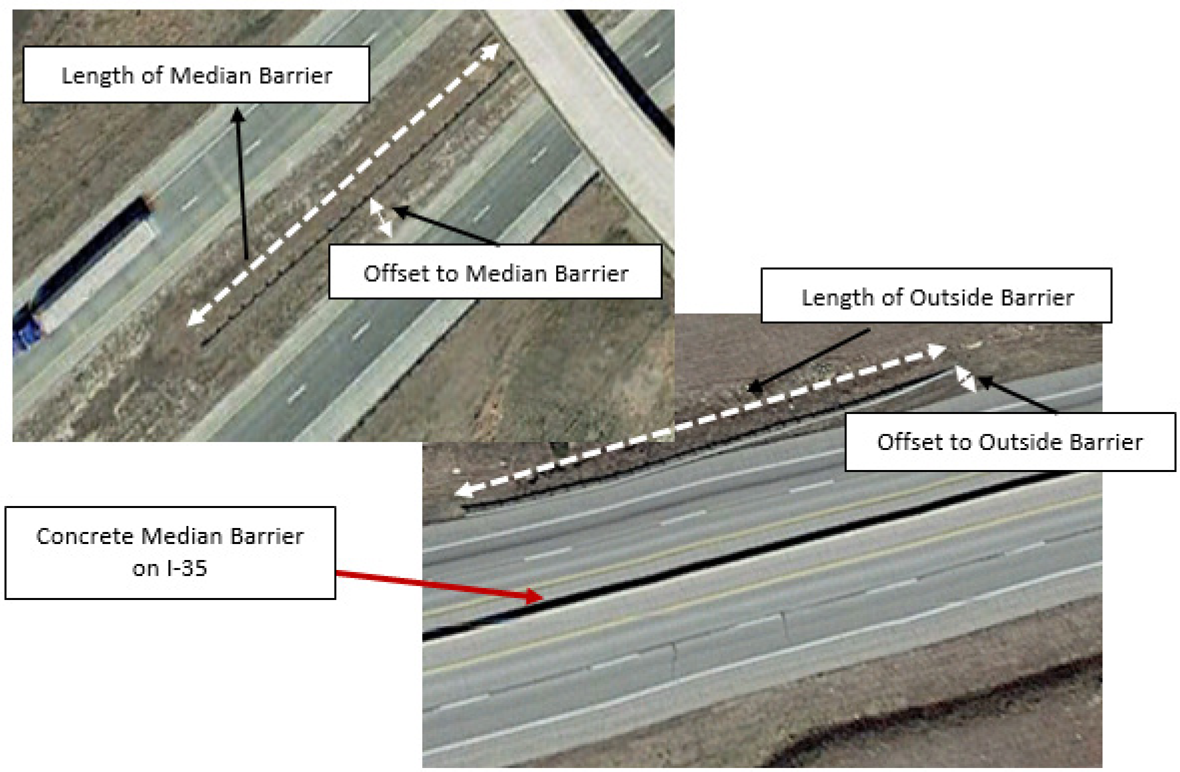


Figure 3.8: Length of and Offset to Median/Outside Barrier

3.5.1.12 Clear Zone Width

Clear zone width is the distance from the edge of the traveled way including the outside shoulder to vertical obstructions, such as fences, utility poles, or non-traversable slopes (AASHTO, 2014). Figure 3.9 illustrates the extent of a clear zone (Whc) as defined in the HSM. It should be noted that lone trees are not considered as vertical obstructions. Additionally, outside barriers are also not considered as vertical obstructions as the effect of those are covered in freeway crash modification factors. Like other variables, the average length is calculated if the clear zone width varies along different points of the segment.

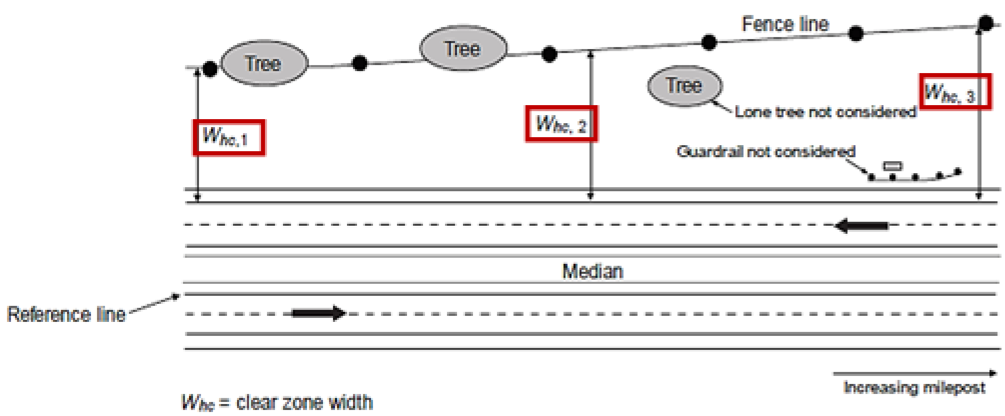
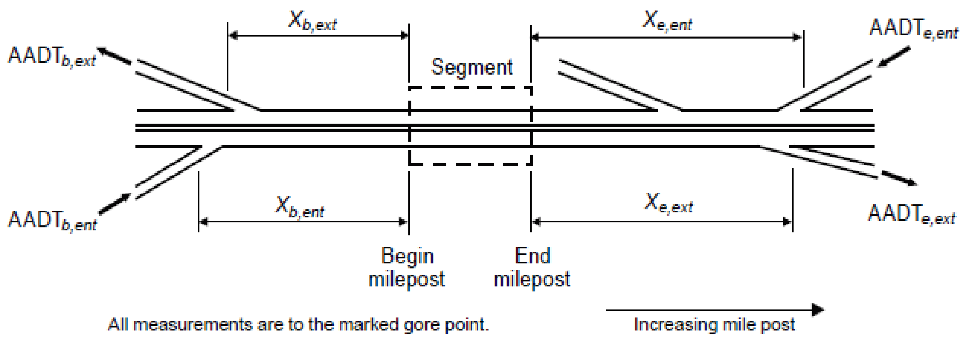


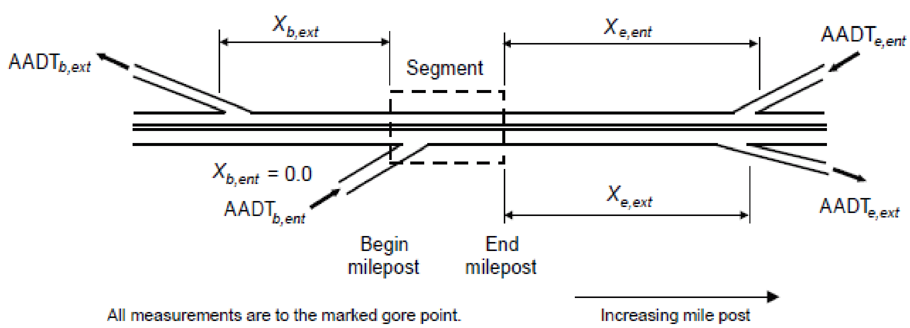
Figure 3.9: Measurement of the Clear Zone Width
AASHTO (2014)

3.5.1.13 Distance to the Nearest Entrance/Exit Ramp

The distance to the nearest entrance or exit ramp should be measured from the edge of the segment to the nearest entrance or exit ramp in both directions of travel. Figure 3.10(a) displays an example estimation of distances to the nearest entrance/exit ramps when the segment is located externally to all ramps. For the increasing milepost direction, the distance (Xb,ent) is measured from the beginning milepost to the upstream entrance ramp, and the distance (Xe,ext) is measured from the end milepost to the downstream exit ramp. For the decreasing milepost direction, the distance, Xb,ext is measured from the beginning milepost to the upstream exit ramp, and the distance, Xe,ent is measured from the end milepost to the downstream entrance ramp. Figure 3.10(b) displays an example estimation of distances to the nearest entrance/exit ramps when the segment is located on a ramp. In this study, distances to the nearest exit and entrance ramps were measured using the measurement tool in ArcGIS (Esri, 2012).



(a) Distance when all ramps are located externally to the segment



(b) Distance when one ramp is located in the segment

Figure 3.10: Distance to Nearest Entrance/Exit Ramp
AASHTO (2014)

3.5.1.14 Proportion of High Volume

The proportion of high volume is the proportion of freeway AADT volume that occurs during hours when the volume exceeds 1,000 veh/hr/ln (AASHTO, 2014). This study used the default equation given in the HSM to calculate the proportion of high volume.

3.5.2 Ramp Segment Calibration

More than 90% of geometric data required for the ramp segment calibration were extracted using Google Earth. Similar to freeway segments, area type for ramp segments was identified using the CANSYS database. Segment lengths of ramps were extracted from the

Highway Performance Monitoring System (HPMS) GIS shapefiles (FHWA, 2016). To obtain the radii for horizontal curves on ramps, KDOT's built-in ArcGIS curve tool was run for the latest ramp GIS shapefile (KDOT, 2015). All other geometric variables, such as number of through lanes, lane width, paved left/right shoulder width, length of and offset to left/right side barrier, presence of lane add or drop, presence of speed-change lane, and presence and length of weaving section, were collected using Google Earth. Figure 3.11 and Figure 3.12 show the measurement of length and offset to outside/inside barriers and measurement of lane width and paved shoulder width on ramps using Google Earth, respectively.

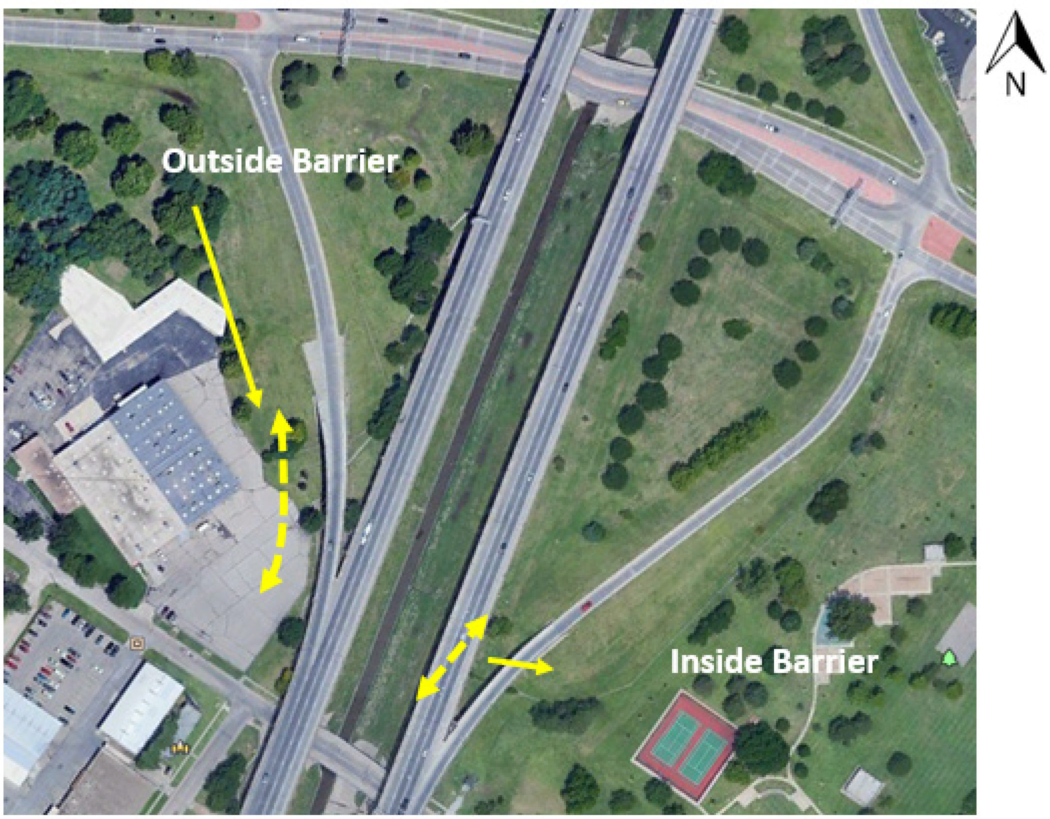


Figure 3.11: Length of and Offset to Outside/Onside Barrier on Ramps using Google Earth

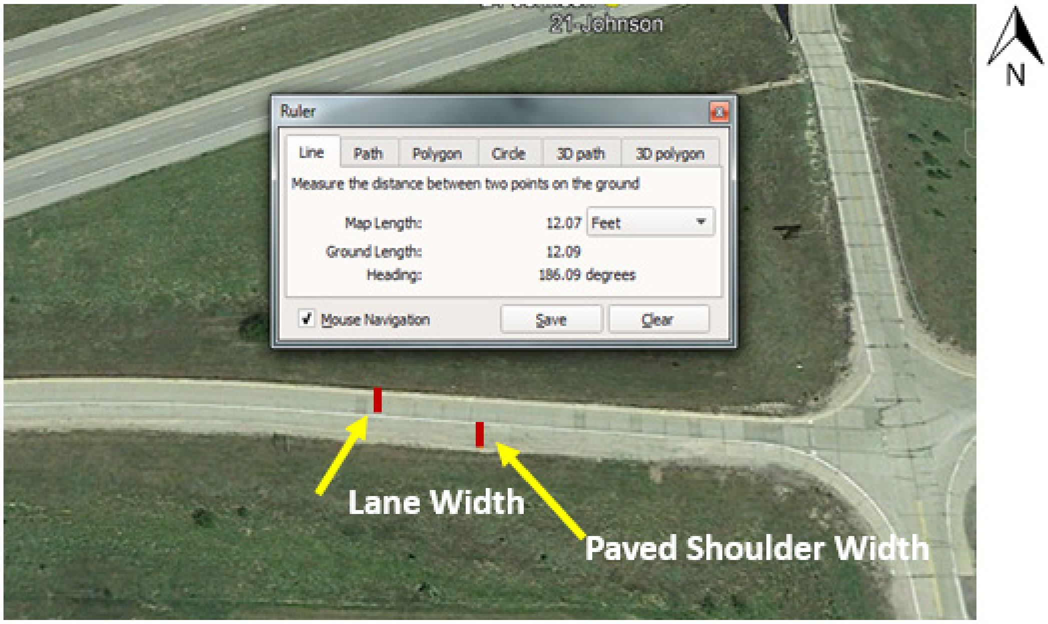


Figure 3.12: Lane Width and Paved Shoulder Width Dimensions using Google Earth

3.5.3 Crossroad Ramp Terminal Calibration

Similar to ramp segments, more than 90% of geometric data required for the crossroad ramp terminal calibration were extracted from Google Earth. Ramp terminal configuration, number of through lanes on each crossroad approach, number of lanes on the exit ramp, distance to the next public street intersection, distance to adjacent crossroad ramp terminal, crossroad median width and left-turn width, number of crossroad approaches with right-turn channelization, presence of exit ramp right-turn channelization, and presence of non-ramp public street leg were some of the data elements collected using Google Earth. Other variables, such as type of traffic control, control for exit ramp right-turn movement, number of crossroad approaches with left- and right- turn lanes, number of unsignalized public street approaches to the crossroad leg outside of the interchange, number of unsignalized driveways on the crossroad leg outside of the interchange, and number of crossroad approaches with protected-only left-turn operations were obtained from the Google street-view mode. Figure 3.13 shows measurement of distance to the adjacent ramp terminal and public street using Google Earth. The skew angle was measured inserting a "compass tool" to Google Earth as shown in Figure 3.14.

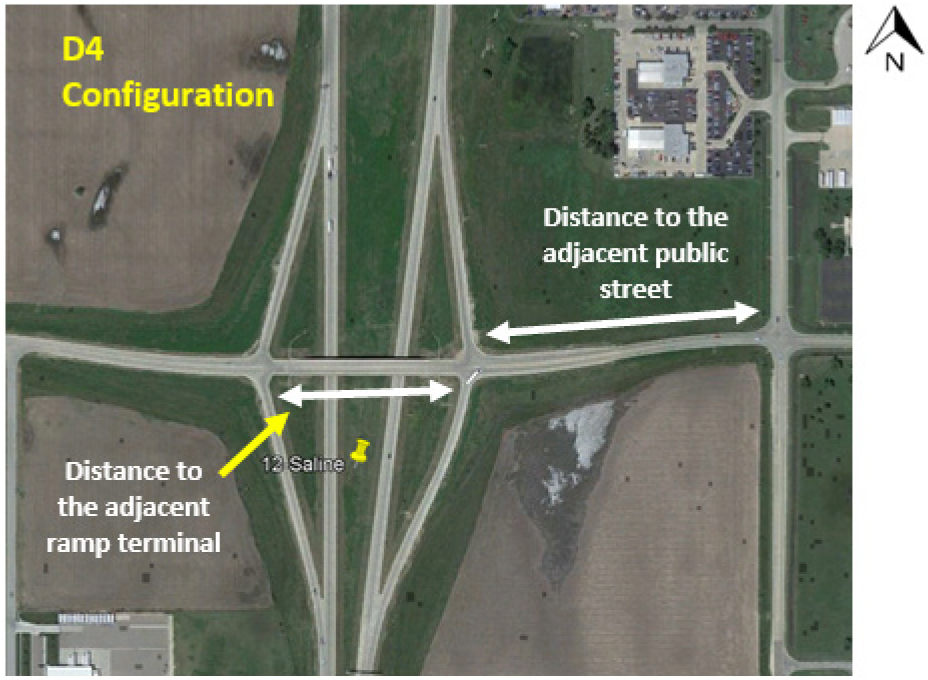


Figure 3.13: Distance to Adjacent Ramp Terminal/Public Street using Google Earth

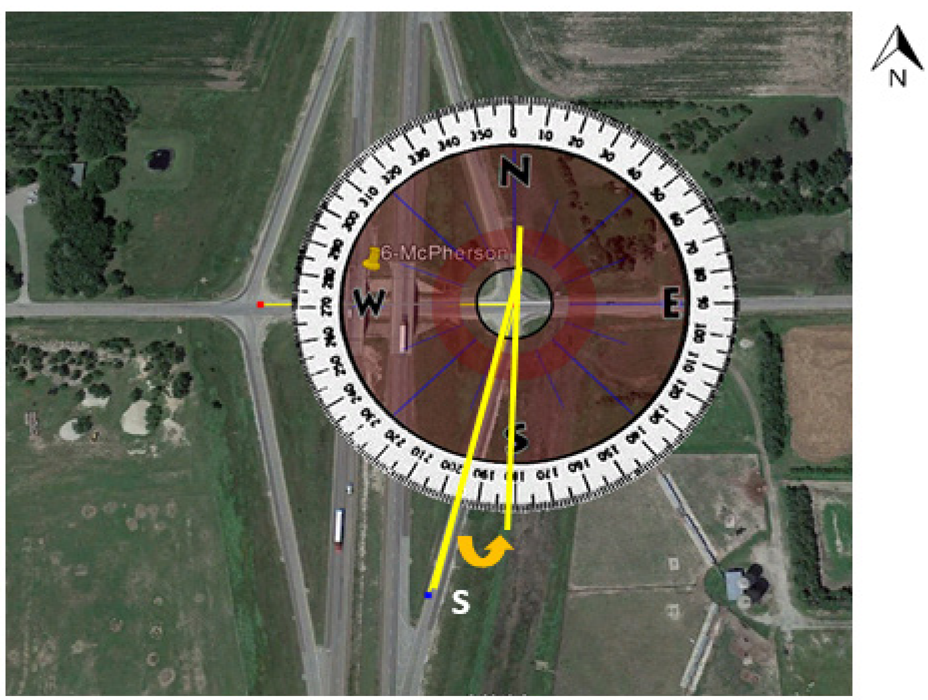


Figure 3.14: Measuring Skew Angle using Google Earth

All data elements discussed in this section correspond to the data elements provided in Table 3.2, Table 3.3, and Table 3.4, which were needed to perform freeway facility and ramp facility calibration.

3.6 Extraction of AADT Volume of Freeways, Ramps, and Crossroads

Even though the CANSYS database included AADTs of all freeways in Kansas for the entire study period, AADT volumes for ramps were not available in CANSYS. Ramp AADT volumes and crossroad AADT volumes were extracted from the HPMS GIS shapefiles. The HPMS is a national-level highway information system that contains data on the condition, performance, use, and operating characteristics of selected highways in the US (FHWA, 2016). Each year, KDOT submits condition, extent, and performance data of selected state highways in Kansas to the HPMS administrated by the FHWA. In addition, KDOT maintains two separate GIS shapefiles for ramp-related data, which includes AADT volume and segment lengths of ramps in Kansas as shown in Figure 3.15.

(a) Ramp (b) Ramp + Speed-Change

Figure 3.15: GIS Shapefiles for Kansas Ramps

3.7 Segmentation

According to the HSM, a new homogenous segment should begin when a change occurs in at least one of the freeway characteristics, such as number of through lanes, lane width, outside shoulder width, inside shoulder width, median width, ramp presence, or clear zone width (AASHTO, 2014). In fact, each row of the CANSYS database represents a homogenous segment indicating that geometric attributes of each segment were consistent in between given milepost locations.

3.7.1 Segment Reduction Procedure

As the CANSYS database is comprised of geometric and condition data for all I, K, and US roadways in Kansas, the main goal was to isolate freeway segments that met HSM freeway definition criteria. The CANSYS database includes records of homogeneous segments on both sides of a roadway (right side and left side). For example, as shown in Figure 3.16, the west-bound (WB) lane of I-70 is the left side and the east-bound (EB) lane of I-70 is the right side of the freeway. Similarly, for roadways that run in north-south directions, the south-bound (SB) lane is the left side and the north-bound (NB) lane is the right side of the freeway.

Figure 3.16: Example of Identification of Freeway Sides Based on CANSYS

3.7.2 Freeway Segment Database

The freeway segment database was created to perform the freeway segment calibration in Kansas with respect to HSM calibration criteria. This database was created by reducing segments from the original CANSYS database. Table 3.5 lists the steps followed in selecting freeway segments that meet the HSM definition. At Step 1, all fully access-controlled segments were filtered out from the CANSYS. In Step 2, the filtered segments were further screened by median type, presence of toll plazas, HOV lanes, reversible lanes, ramp metering, presence of an intersection within 0.5 miles, and all other limitation criteria listed in the HSM (AASHTO, 2014). Step 3 was conducted to divide the segments into 1-mile sections, as the HSM requires the segments to be 0.1 to 1.0 mile in length for the calibration.

Table 3.5: Segment Reduction Procedure in Developing Freeway Segment Database

Step No.	Action Taken	Number of Segments		
		Right Side	Left Side	Total
Step 1	Extract fully access-controlled segments from CANSYS database	2,700	2,013	4,713
Step 2	Select freeway segments by HSM definition	1,552	1,189	2,741
Step 3	Divide into 1-mile sections	2,484	2,127	4,611
Step 4	Select freeway segments associated with ramps (Speed-change Lane Database)	1,047	918	1,965
Step 5	Remove the number of segments obtained in Step 4 from the Step 3 (Initial Freeway Segment Database)	1,433	1,204	2,637
Step 5.1	Discard 2 lane, 5 lane, & 7 lane segments	1,417	1,201	2,618
Step 5.2	Select segments with speed limit ≥ 65mph	1,367	1,174	2,541
Step 5.3	Select segments ≥ 0.1 mile in length	1,131	995	2,126
Step 5.4	Select segments with matching mileposts on both sides of freeway (Final Freeway Segment Database)	1,133		

3.7.3 Speed-Change Lane Database

The speed-change lane database was created to perform the speed-change lane calibration in Kansas with respect to HSM calibration criteria. Preparation of the speed-change lane database was more challenging compared to the freeway segment database, because the CANSYS database did not generate a new homogeneous segment with a ramp being present.

Therefore, when creating the freeway segment database in Step 4 as shown in Table 3.5, freeway segments connected to ramps were selected and those segments were further filtered with respect to the HSM criteria to create the speed-change lane database.

The two ramp shapefiles shown in Figure 3.15 were manually edited by adding two fields (columns) named, “ramp side” and “EN/EX.” The “ramp side” field was recorded as the left or right side, depending on the side on which the ramp was located on the freeway. If the ramp was connected to an entrance-related speed-change lane along the freeway, “En” was recorded in the “EX/EN” field, and if the ramp was connected to an exit speed-change lane along the freeway, “Ex” was recorded in the “EN/EX” field. Table 3.6 indicates the steps followed to obtain the speed-change lane database after isolating freeway segments connected to ramps. Step 4 and Step 5 in Table 3.6 were carried out to isolate speed-change lane sites from freeway segments connected to ramps. In order to isolate speed-change lane sites, endpoints (gore and taper) were created for both HPMS ramp GIS shapefiles (ramps shapefile and ramps+speed-change shapefile) in ArcGIS as shown in Figure 3.17.

Table 3.6: Segment Reduction Procedure in Developing Speed-Change Lane Database

Step No.	Action Taken	Number of Segments		
		Right Side	Left Side	Total
Step 1	Select freeway segments connected to ramps (Step 4 in Table 3.3)	1,047	918	1,965
Step 2	Discard 2 lane, 5 lane, & 7 lane segments	1,030	909	1,939
Step 3	Select segments with the speed limit ≥ 65mph	920	833	1,753
Step 4	Split at the gore point	514	477	991
Step 5	Split at the taper point and separate speed-change lane sites	446	416	865
Step 6	Select entrance related speed-change lanes (manually)	168	187	355
Step 7	Select exit related speed-change lanes (manually)	192	178	370
Step 8	Select entrance related segments between 0.04–0.30 miles in length (Final EN speed-change lane database)	166	185	351
Step 9	Select exit related segments between 0.02–0.30 miles in length (Final EX speed-change lane database)	190	176	366

Figure 3.18 graphically explains the procedure for isolating a sample speed-change lane site using ArcGIS. At first, freeway segments connected to ramps were split at the gore point. Then, the segment obtained in the previous step was further split at the taper point. After implementing these two steps, the segment left out in between the gore and taper points is the speed-change lane. Similarly, this procedure was carried out for all freeway segments connected to ramps on both the left and right sides of the roadway. Once the speed-change lane sites were extracted for both sides of the freeway, a new column was added, called "EN/EX" to the speed-change lane database. If the speed-change lane was entrance-related (entrance to the freeway), "En" was recorded in the "EX/EN" field, and if speed-change lane is exit-related (exit from the freeway), "Ex" was recorded in the "EN/EX" field. This process was carried out manually, and Step 6 and Step 7 in Table 3.6 provide the resulting entrance-related and exit-related speed-change lanes. It should be noted that, when processing Step 6 and Step 7, more than 100 segments were removed in Step 5 because those homogeneous segments ended up in between gore and taper points according to the original CANSYS segmentation process. Afterwards the minimum length for an entrance-related speed-change lane was considered as 0.04 miles, and the minimum length for an exit-related speed-change lane was considered as 0.02 miles.

Figure 3.17: Creating Endpoints in Ramp GIS Shapefiles

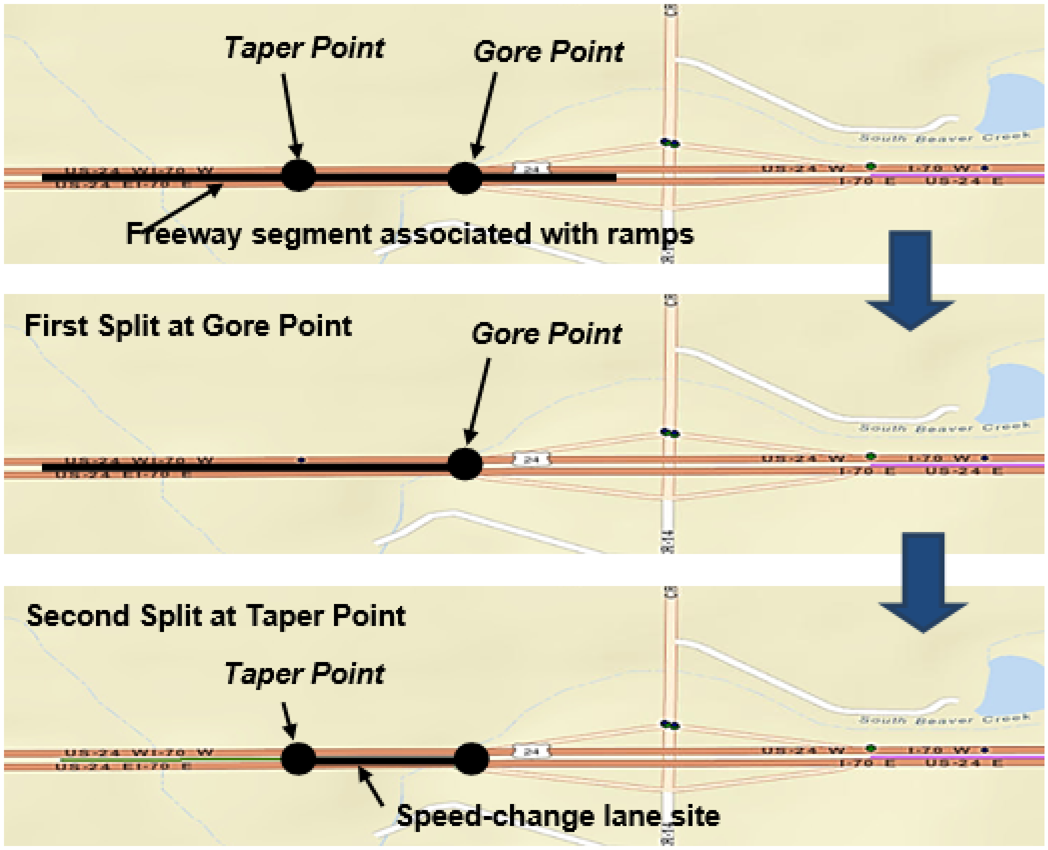


Figure 3.18: Splitting Segments at Taper and Gore Points to Obtain Speed-Change Lane Sites

3.7.4 Ramp Segmentation

According to the HSM, ramp segmentation occurs when a change occurs in at least one of the characteristics, such as number of through lanes, lane width, right shoulder width, left shoulder width, merging ramp, or diverging ramp. Figure 3.19 shows a guideline provided in the HSM to determine the number of lanes in ramp segments. This is an exit ramp where the most right end of Segment 3 ends at the crossroad ramp terminal.

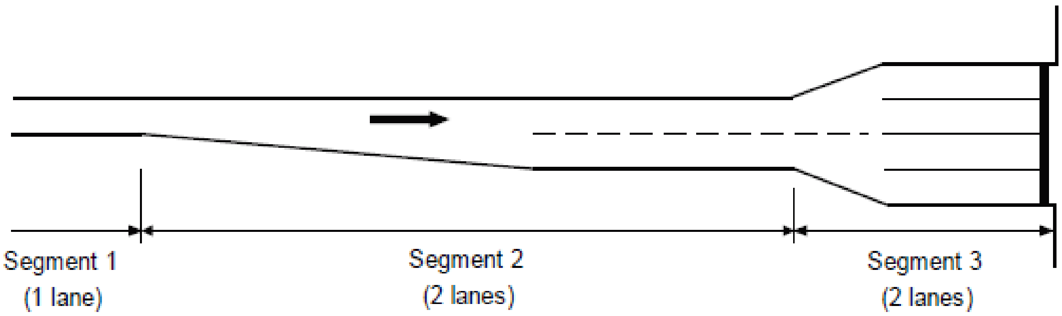


Figure 3.19: Number of Lanes in Ramps
AASHTO (2014)

This study did not consider ramp segmentation when a change in number of lanes occurred due to the inability to locate crashes at exacts location on ramps. Nevertheless, if any ramp had three-quarters of its length as 2-lanes, it was considered a 2-lane ramp, and similarly, if any ramp had three-quarters of its length as 1-lane, it was considered a 1-lane ramp.

3.8 Site Selection

3.8.1 Freeway Segment and Speed-Change Lane Calibration Sample

Based on the most recent literature, it seemed that 30 to 50 sites initially suggested by the HSM were not adequate to estimate an accurate calibration factor. Therefore, a more reasonable sample size was determined with a 95% confidence level using Equation 3.2. Shin et al. (2015) and Kim et al. (2015) used this approach to calculate a minimum sample size with respect to a desired confidence level.

$$\boldsymbol{n} = {}^{\boldsymbol{n_0 N}}\!/_{[\boldsymbol{n_0} + (\boldsymbol{N} - \boldsymbol{1})]} \qquad \textbf{Equation 3.2}$$

$$\boldsymbol{n_0} = \boldsymbol{P}\,(\boldsymbol{1} - \boldsymbol{P})\left(\frac{Z}{e}\right)^{\boldsymbol{2}} \qquad \textbf{Equation 3.3}$$

Where:

n	=	Minimum sample size,
Z	=	Area under normal curve to the preferred confidence level,
N	=	Population,
P	=	True population, and
e	=	Margin of errors.

This study considered a 95% confidence level with a 5% margin of error and 50% of the true population. Table 3.7 provides calculated minimum sample sizes for all freeway facility types using Equation 3.2. Considering freeway segments, a minimum sample of 446 sites was required to perform the calibration. After computing the minimum sample size, Hawth's Analysis Tools for ArcGIS (Version 3.27) was used to create a random sample in ArcGIS (Esri, 2012; Beyer, 2004). Even though the minimum sample size required was 446, the study team used data from 521 segments in total for calibration. In the case of speed-change lanes, the entire population of 351 entrance-related speed-change lanes and 366 exit-related speed-change lanes were used for the calibration. The reason for employing the entire population for speed-change lane calibration was that the exit-related speed-change lanes did not fulfill the HSM minimum crash threshold of 100 crashes per year with a minimum sample size of 187 sites. In addition, most of the recent literature related to HSM calibration mentioned that because data systems and number of observed crashes vary considerably across states, sometimes a larger sample size may be required (Alluri et al., 2016; Bahar, 2014; Banihashemi, 2012; Kim et al., 2015; Trieu et al., 2014). Figure 3.20 and Figure 3.21 provide the geographical distribution of freeway segments and speed-change lanes utilized in this calibration study, respectively. In fact, both the freeway segment sample and speed-change lane sample used for the calibration were well distributed on the state network.

Table 3.7: Minimum Sample Size Requirement at 95% Confidence Level for Freeway Segments and Speed-Change Lanes

Facility Type	Total Population	95% CI	No. of Sites Used for Calibration
		Min. Sample Size	
Freeway Segments			
Rural 4-lane freeways (R4F)	896	270	338
Rural 6-lane freeways (R6F)	18	18	18
Urban 4-lane freeways (U4F)	178	122	142
Urban 6-lane freeways (U6F)	35	35	20
Urban 8-lane freeways (U8F)	4	4	3
Total Freeway Segments	**1,133**	**446**	**521**
Speed-Change Lanes			
Speed-change lanes entering rural 4-lane freeway (R4SCen)	200	132	200
Speed-change lanes entering urban 4-lane freeway (U4SCen)	108	84	108
Speed-change lanes entering urban 6-lane freeway (U6SCen)	35	32	35
Speed-change lanes entering urban 8-lane freeway (U8SCen)	8	8	8
Speed-change lanes exiting rural 4-lane freeway (R4SCex)	215	138	215
Speed-change lanes exiting urban 4-lane freeway (U4SCex)	114	99	114
Speed-change lanes exiting urban 6-lane freeway (U6SCex)	31	29	31
Speed-change lanes exiting urban 8-lane freeway (U8SCex)	6	6	6
Total Entrance Speed-change lanes (SCen)	**351**	**184**	**351**
Total Exit Speed-change lanes (SCex)	**366**	**187**	**366**

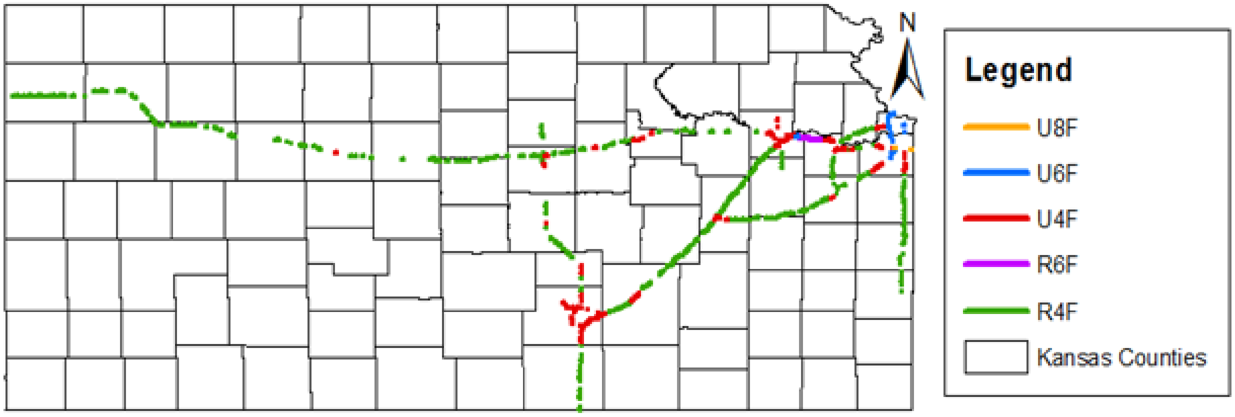


Figure 3.20: Freeway Segment Calibration Sample

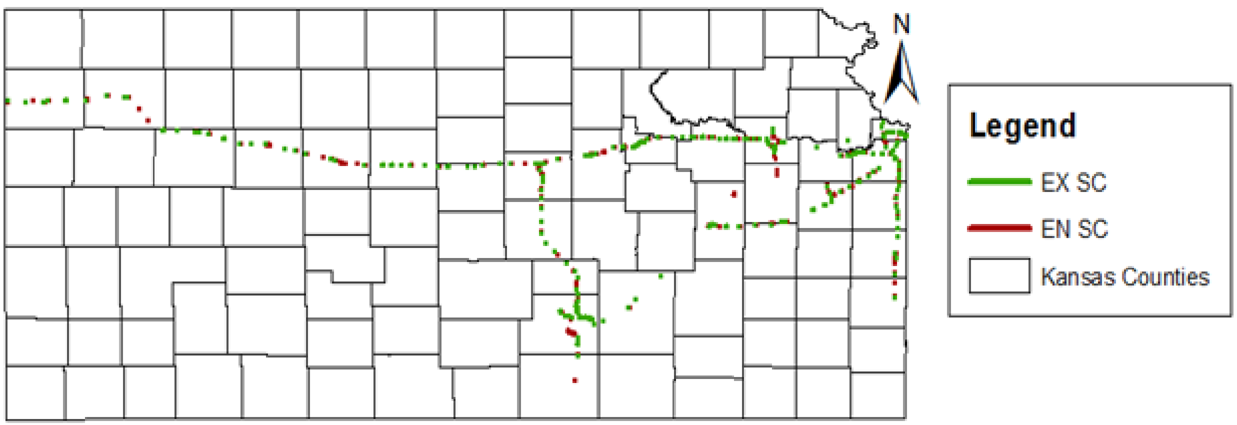


Figure 3.21: Speed-Change Lane Calibration Sample

3.8.2 Ramp Segment and Crossroad Ramp Terminal Calibration Sample

Figure 3.22 shows 33 counties where freeways under study pass through in the state of Kansas. Out of those 33 counties, 15 were randomly selected using the Microsoft Excel random generator tool. Ramp segments and crossroad ramp terminals from those randomly selected 15 counties, namely, Johnson, Shawnee, Wabaunsee, Douglas, McPherson, Harvey, Sedgwick, Saline, Dickinson, Ellsworth, Butler, Miami, Russell, Gove, and Geary were used for the calibration. Table 3.8 provides the number of sites used for the ramp segment, stop-controlled crossroad ramp terminal, and signal-controlled crossroad ramp terminal calibrations in Kansas. Accordingly, 184 entrance ramps, 156 exit ramps, 120 stop-controlled crossroad ramp terminals, and 74 signal-controlled crossroad ramp terminals were used. Figure 3.23, Figure 3.24, and Figure 3.25 provide the geographical distribution of ramp segments, signal-controlled crossroad ramp terminals, and stop-controlled crossroad ramp terminals utilized in this calibration study, respectively. Like freeway facilities, samples selected for the ramp segment and crossroad ramp terminal calibration were also well distributed on the state network.

Table 3.8: Selected Ramp Segments and Crossroad Ramp Terminals for the Calibration

Facility Type	No. of Sites Used for Calibration
Ramps	
Urban 1-lane entrance ramps (U1EN)	112
Urban 2-lane entrance ramps (U2EN)	10
Rural 1-lane entrance ramps (R1EN)	62
Urban 1-lane exit ramps (U1EX)	77
Urban 2-lane exit ramps (U2EX)	19
Rural 1-lane exit ramps (R1EX)	60
Total Entrance Ramps	**184**
Total Exit Ramps	**156**
Stop-Controlled Crossroad Ramp Terminals	
Rural D4 stop-controlled terminals (RD4ST)	47
Rural A2 stop-controlled terminals (RA2ST)	1
Rural B2 stop-controlled terminals (RB2ST)	1
Urban D4 stop-controlled terminals (UD4ST)	55
Urban A2 stop-controlled terminals (UA2ST)	4
Urban B2 stop-controlled terminals (UB2ST)	2
Urban D3en stop-controlled terminals (UD3enST)	4
Urban D3ex stop-controlled terminals (UD3exST)	6
Total Stop-controlled Crossroad Ramp Terminals	**120**
Signal-Controlled Crossroad Ramp Terminals	
Urban D4 signal-controlled terminals (UD4SG)	47
Urban A2 signal-controlled terminals (UA2SG)	9
Urban A4 signal-controlled terminals (UA4SG)	1
Urban B2 signal-controlled terminals (UB2SG)	10
Urban D3en signal-controlled terminals (UD3enSG)	2
Urban D3ex signal-controlled terminals (UD3exSG)	5
Total Signal-controlled Crossroad Ramp Terminals	**74**

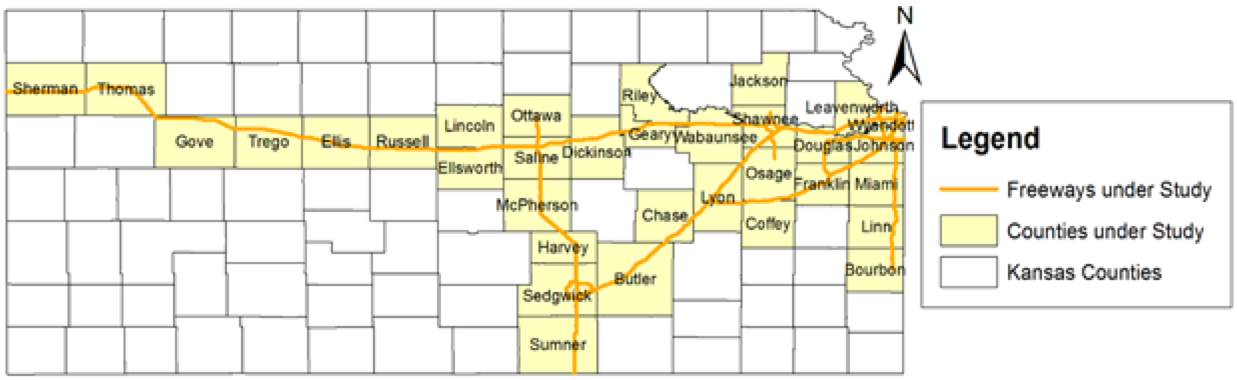


Figure 3.22: Counties where Freeways under Study Pass Through in Kansas

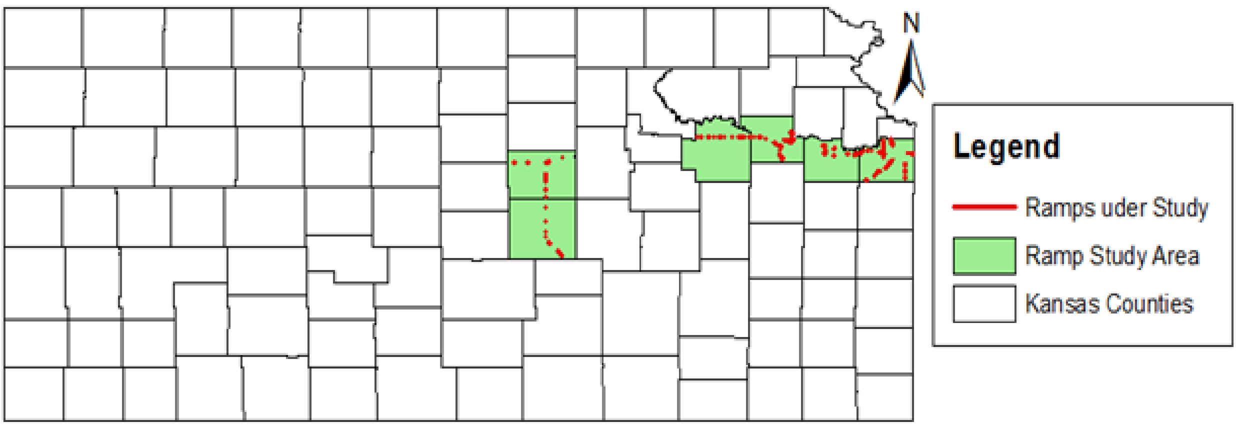


Figure 3.23: Ramp Segment Calibration Sample

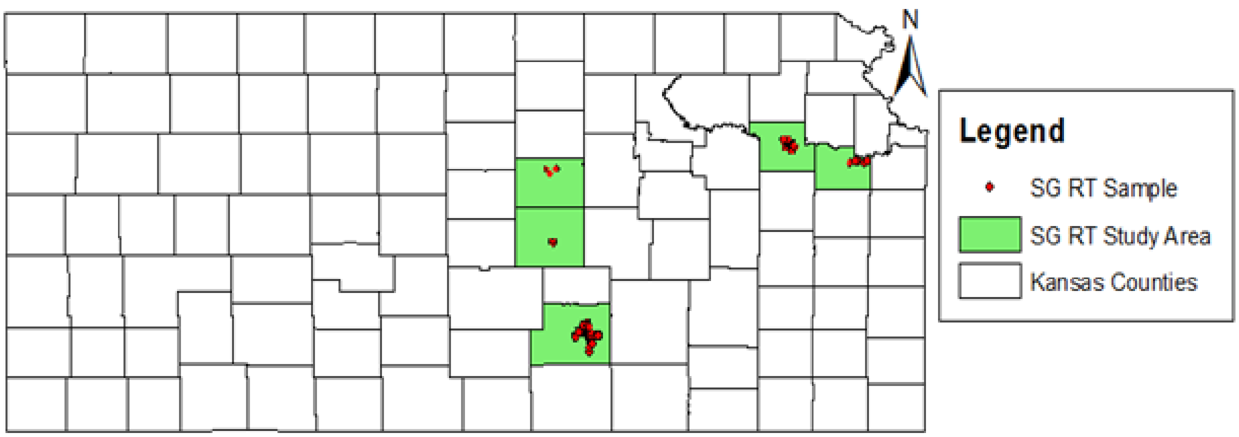


Figure 3.24: Signal-Controlled Crossroad Ramp Terminal Calibration Sample

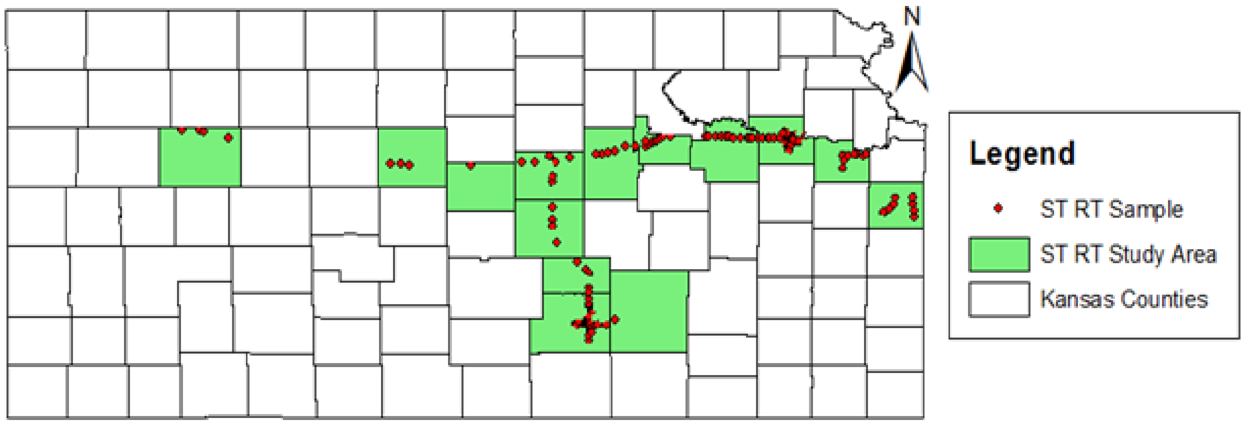


Figure 3.25: Stop-Controlled Crossroad Ramp Terminal Calibration Sample

3.9 Reported Crashes

3.9.1 Crash Severity

Five types of injury severity were reported in Kansas: F - fatal (K), D - disabled (A), I - injury (B), P - possible injury (C), and N - not injured (O). The "Accident Severity" field in the ACCIDENT_SUMMARY table of the KCARS database records these five injury severities under three major levels of crash severity, as described in the following sections.

3.9.1.1 Fatal (F) crashes

If a crash results in the death of one or more persons at the time of the accident, or within 30 days of the accident, it is classified as a fatal crash. If a person dies because of a medical condition and not as a result of the accident, the injury severity is recorded as per the accident repercussion (KDOT, 2014).

3.9.1.2 Injury (I) crashes

KDOT classifies injury severity into three severity levels (KDOT, 2014):

1. Disabled (incapacitating) – Any injury, other than a fatal injury, that prevents the injured person from driving, walking, or continuing regular activities that he/she was capable of performing before the injury occurred.

2. Injury (non-incapacitating) – Any injury that is not fatal or disabling, which is evident to observers at the sight of the crash. Inclusions: lump on head, abrasions, bruises, and minor lacerations.
3. Possible injury – Any reported or claimed injury that is not fatal, disabling, or incapacitating. Inclusions: momentary unconsciousness, limping, complaint of injuries not evident, nausea, and hysteria.

3.9.1.3 Property Damage Only (PDO) crashes

If a crash causes damage to public or private property higher than a $1,000 threshold with no injuries reported, it is identified as a PDO crash (KDOT, 2014). According to the HSM, crash severities for freeway and ramp calibration models are considered as Fatal and Injury (FI) crashes, and Property Damage only (PDO) crashes.

3.9.2 Extracting Crash Data from KCARS Database for Freeway Segments and Speed-Change Lanes

The KCARS database consists of information on all crashes occurring on Kansas roadways reported as accidents. It is a Microsoft Access-based database consisting of several tables carrying detailed information about the crash such as location, severity, functional class, manner of collision, time and data of crash, weather and light conditions, road surface type, road conditions, etc. This database is coded to comply with the Kansas Motor Vehicle Accident Report (KDOT, 2014). Crash records from different tables in the database could be linked with use of the accident key, a unique ID assigned to each crash. In this study, a query was designed as shown in Figure 3.26 linking three main tables: ACCIDENTS, ACCIDENT_SUMMARY, and ACCIDENT_CANSYS by the accident key to obtain data on crashes on freeways with respect to three levels of crash severity. Once crash data for freeways were extracted from the KCARS database, these crashes were mapped in ArcGIS using the latitude and longitude of each crash location (Esri, 2012). Then, the observed crashes for each selected freeway segment or speed-change lane were counted for the study period. In case of speed-change lanes, crashes were mapped according to the left and right side of the freeway as explained in Section 3.7.1.

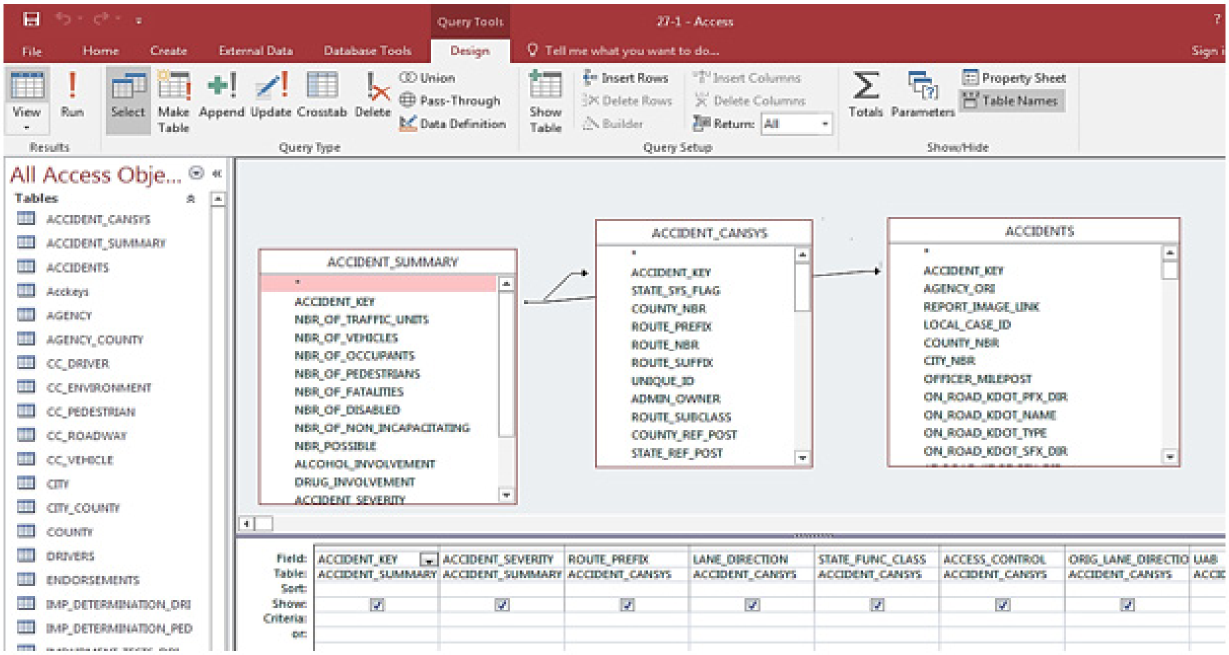


Figure 3.26: Snip of Query Design in KCARS Database

Figure 3.27 indicates crashes on fully access-controlled highways in Kansas from 2013 to 2015, Figure 3.28 provides assignment of crashes on sample freeway segments, and Figure 3.29 provides assignment of crashes on sample entrance and exit speed-change lanes.

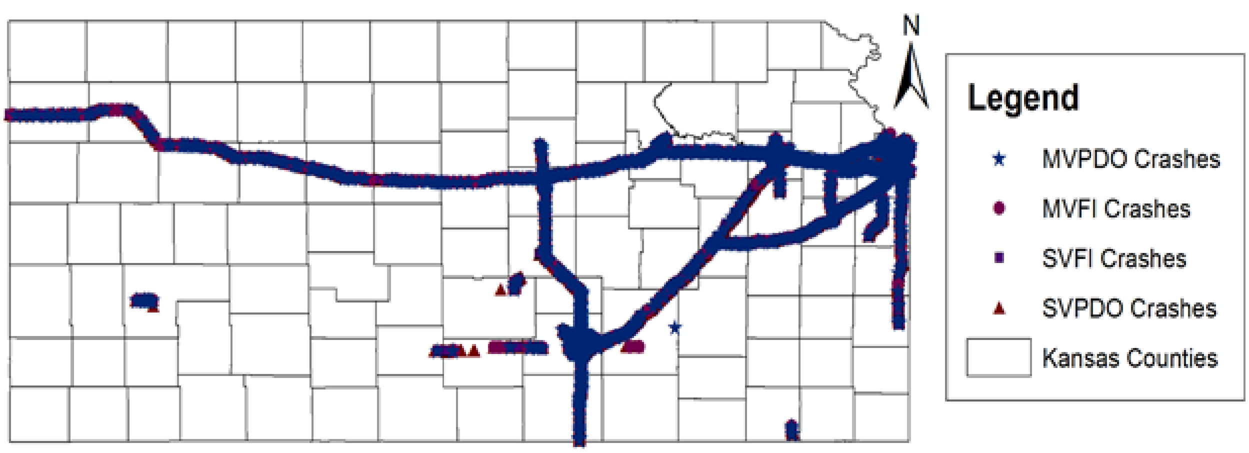


Figure 3.27: Crashes on Freeways under Study (2013–2015)

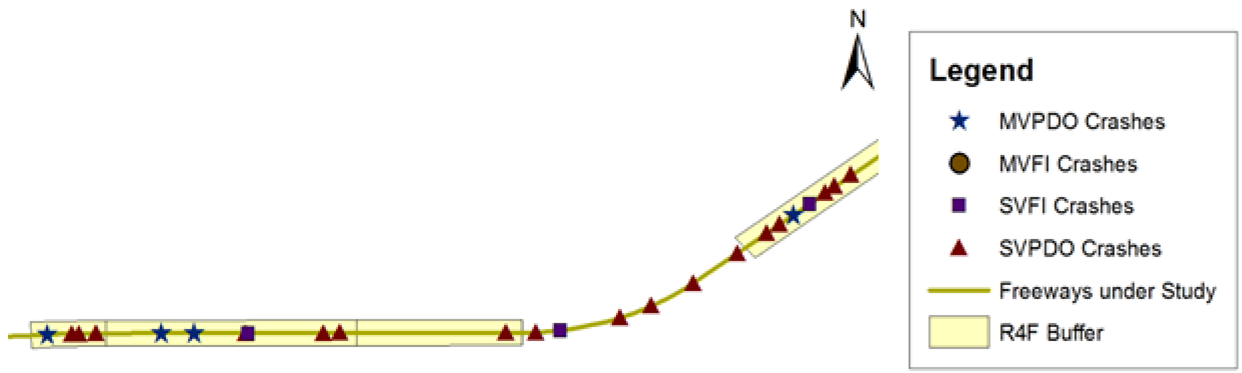


Figure 3.28: Crashes on Sample Freeway Segments

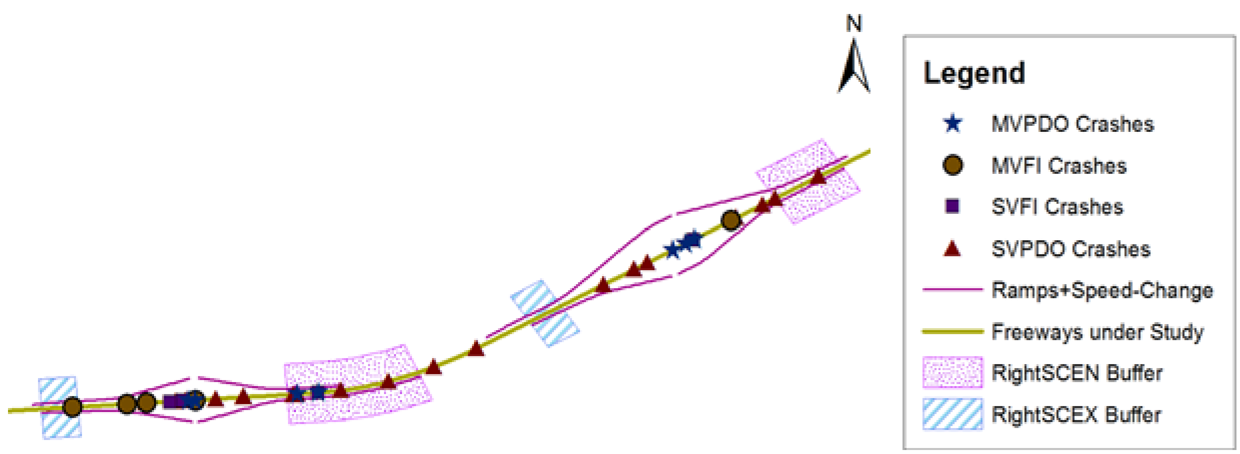


Figure 3.29: Crashes on Sample EN/EX Speed-Change Lanes (East-Bound)

3.9.3 Crash Extraction for Ramp Segments and Crossroad Ramp Terminals

Crashes for ramp segments and crossroad ramp terminals were identified by manually reviewing all interchange-related crash reports from 2014 to 2016. Even though crashes for ramps were indirectly included in the KCARS database, after an in-depth investigation it was observed that most crashes recorded as ramp crashes were actually crossroad ramp terminal crashes or speed-change lanes. Therefore, this study manually reviewed 13,730 interchange-related crash reports in total from 15 counties as mentioned in Section 3.8.2. When reviewing interchange-related crash reports manually, the collision diagram and crash description were thoroughly studied to identify crashes located on ramp segments and crossroad ramp terminals. According to the HSM, driveway- and public street-related crashes on the crossroad within 250 ft of the crossroad ramp terminal should be assigned to the crossroad ramp terminal (AASHTO, 2014, pp, Appendix B-20).

3.10 Allocation of Crashes and Site Characteristics for Freeway Segments, Speed-Change Lanes, Ramp Segments, and Crossroad Ramp Terminals

This section explains allocation of crashes and site characteristics of freeway segments, speed-change lanes, ramp segments, and crossroad ramp terminals utilized for calibration. Site characteristics for freeway segments, speed-change lanes, and ramp segments include the AADT of the segment and segment length of the facility type. Site characteristics for crossroad ramp terminals include AADT of inside/outside legs of the crossroad, and AADT of associated exit and entrance ramps. The inside leg is the crossroad between ramps and the outside leg is the crossroad outside of the interchange as illustrated in Figure 3.30.

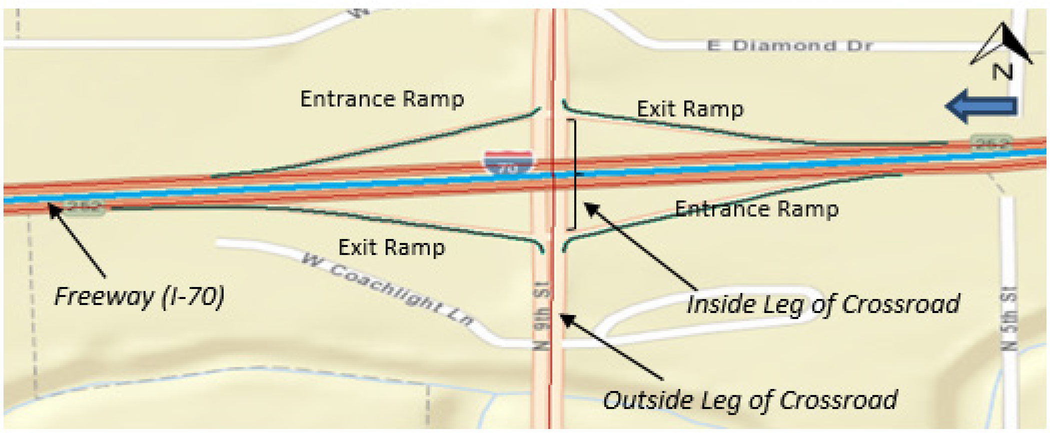


Figure 3.30: Illustration of Inside/Outside Leg of Crossroad

Table 3.9 shows segment characteristics and crash allocations of freeway segments considered for the calibration. Roughly, 77% of the total length of freeway segments, 55% of freeway crashes, and 52% of freeway fatal and injury crashes occurred in rural settings. The rural 4-lane freeway is the most dominant freeway type in the Kansas freeway segment sample, with 338 segments consisting of 194.8 miles in length and 1,574 reported crashes from 2013 to 2015. The AADT for the total freeway segment sample fluctuated between 4,260 and 145,000 vehicles per day (vpd), whereas the segment length varied from 0.10 to 1.00 mile, which is compatible with the HSM recommendations. Urban 8-lane freeways reported the highest crash rate per 100

Million Vehicle Miles Traveled (MVMT) and urban 4-lane freeways reported the second highest crash rate per 100 MVMT.

Table 3.9: Crash Allocation and Segment Characteristics of the Freeway Segment Sample

Freeway Facility (Freeways)	Freeway AADT (vpd)			Segment Length (miles)				FI Crashes (2013–2015)	PDO Crashes (2013–2015)	Crash Rate per 100 MVMT
	Min	Max	Average	Min	Max	Average	Total			
R4F	4,260	36,500	13,026	0.11	1.00	0.58	194.84	285	1,289	56.64
R6F	37,600	41,000	39,067	0.15	1.00	0.62	11.15	62	261	67.70
U4F	6,450	66,300	23,865	0.10	1.00	0.40	56.87	233	972	81.09
U6F	31,100	109,000	67,443	0.10	0.53	0.27	5.42	47	183	57.49
U8F	68,600	145,000	110,967	0.11	0.45	0.14	0.70	33	87	140.56
Total							**268.98**	**660**	**2,792**	**66.03**

Note: FI – Fatal and Injury, PDO – Property Damage Only, vpd – vehicles per day, MVMT – Million Vehicle Miles Traveled, R – Rural, U – Urban

Table 3.10 shows segment characteristics and crash allocations of entrance speed-change lanes considered for the calibration. Approximately 77% of fatal and injury crashes and total entrance speed-change lane crashes are in urban settings. However, only 44% of total entrance speed-change lane lengths are in urban settings. In fact, the number and complexity of the interchanges are higher in urban areas. In contrast, the rural 4-lane entrance speed-change lane is the most prevalent entrance speed-change lane type in this sample. There are 200 rural 4-lane entrance speed-change lane sites comprising 30.5 miles of length. The total segment length of entrance speed-change lanes varied from 0.06 to 0.29 miles, while complying with the HSM recommendation of 0.3 miles as the maximum length to be considered. Urban entrance speed-change lanes recorded the highest crash rates per 100 MVMT.

Table 3.10: Crash Allocation and Segment Characteristics of the Entrance Speed-Change Lane Sample

Freeway Facility (Speed-Change Lanes)	Freeway AADT (vpd)			Segment Length (miles)				FI Crashes (2013–2015)	PDO Crashes (2013–2015)	Crash Rate per 100 MVMT
	Min	Max	Average	Min	Max	Average	Total			
R4SCEn	2,650	31,200	14,053	0.08	0.28	0.15	30.51	36	130	35.36
U4SCEn	6,970	75,600	28,085	0.06	0.29	0.16	17.43	62	180	45.15
U6SCEn	31,100	149,000	71,901	0.06	0.26	0.16	5.70	45	155	44.57
U8SCEn	73,200	152,000	135,821	0.10	0.21	0.14	1.09	28	102	80.19
Total SCEn							**54.73**	**171**	**567**	**45.66**

Note: FI – Fatal and Injury, PDO – Property Damage Only, vpd – vehicles per day, MVMT – Million Vehicle Miles Traveled, R – Rural, U – Urban

Table 3.11 shows segment characteristics and crash allocations of exit speed-change lanes considered for the calibration. Like entrance speed-change lanes, rural 4-lane exit speed-change lanes are the most predominant exit speed-change lane type in the Kansas exit speed-change lane sample. There are 215 rural 4-lane exit speed-change lane sites totaling 18.6 miles in length. Approximately 73% of the fatal and injury crashes, and total exit crashes are in urban settings. However, only 46% of the total exit speed-change lane lengths are present within urban settings. The total segment length of exit speed-change lanes varied from 0.03 to 0.27 miles, while complying with the HSM recommendation of 0.3 miles as the maximum length to be considered for speed-change lanes. Urban 6-lane and urban 8-lane exit speed-change lanes recorded the highest crash rates per 100 MVMT.

Table 3.11: Crash Allocation and Segment Characteristics of the Exit Speed-Change Lane Sample

Freeway Facility (Speed-Change Lanes)	Freeway AADT (vpd)			Segment Length (miles)				FI Crashes (2013–2015)	PDO Crashes (2013–2015)	Crash Rate per 100 MVMT
	Min	Max	Average	Min	Max	Average	Total			
R4SCEx	2,650	36,500	14,283	0.03	0.19	0.09	18.59	33	95	44.03
U4SCEx	7,300	75,600	28,293	0.04	0.27	0.10	11.59	30	127	43.73
U6SCEx	30,300	149,000	69,009	0.05	0.27	0.12	3.66	43	97	50.62
U8SCEx	38,000	155,000	117,167	0.05	0.21	0.10	0.62	18	40	72.92
Total SCEx							**34.46**	**124**	**359**	**48.02**

Note: FI – Fatal and Injury, PDO – Property Damage Only, vpd – vehicles per day, MVMT – Million Vehicle Miles Traveled, R – Rural, U – Urban

Table 3.12 shows segment characteristics and crash allocations of ramp segments used for the calibration. In the Kansas ramp segment sample, 1-lane ramps are more prevalent than 2-lane ramps. There are 184 entrance ramp segments and 156 exit ramp segments totaling 40 and 36.79 miles in length, respectively. Severe injury crashes or FI crashes are not very predominant in the Kansas ramp sample compared to freeway facilities. The majority of exit- and entrance-ramp crashes occurred in urban settings. In the case of ramp segments, urban 2-lane ramps recorded the highest crash rate per 100 MVMT.

Table 3.12: Crash Allocation and Segment Characteristics of the Ramp Segment Sample

Ramp Facility (Ramps)	Segment AADT (vpd)			Segment Length (miles)				FI Crashes (2014–2016)	Total Crashes (2014–2016)	Total Crash Rate per 100 MVMT
	Min	Max	Average	Min	Max	Average	Total			
U1EN	115	20,933	3,873	0.08	0.68	0.23	25.68	11	58	53.26
U2EN	8,805	15,032	12,603	0.12	0.29	0.19	1.91	5	27	102.44
R1EN	14	4,607	694	0.14	0.40	0.20	12.41	1	5	53.01
U1EX	123	22,170	3,084	0.12	0.66	0.24	18.73	9	47	74.32
U2EX	1,780	19,935	5,311	0.12	0.37	0.24	4.54	2	20	75.76
R1EX	22	3,993	627	0.14	0.45	0.23	13.52	-	3	32.31
Total EN Ramps							**40.00**	**17**	**90**	**62.21**
Total EX Ramps							**36.79**	**11**	**70**	**70.76**

Note: FI - Fatal and Injury, PDO – Property Damage Only, vpd – vehicles per day, MVMT – Million Vehicle Miles Traveled, R – Rural, U- Urban

Table 3.13 shows site characteristics and crash allocations of stop-controlled crossroad ramp terminals used for the calibration. There are 102 D4 stop-controlled crossroad ramp terminals and 18 other configurations of stop-controlled ramp terminals in this calibration sample. The majority of severe injury crashes occurred on D4 stop-controlled ramp terminals. There were no rural D3en and D3ex stop-controlled crossroad ramp terminals in counties considered for this ramp terminal calibration. Urban D4 stop-controlled ramp terminals and rural D4 stop-controlled ramp terminals showed the highest total crash rates per Million Entering Vehicles (MEV).

Table 3.13: Crash Allocation and Site Characteristics of the Stop-Controlled Crossroad Ramp Terminal Sample

Ramp Facility (Stop-controlled Ramp Terminals)	Average Crossroad AADT (vpd)		Average Ramp AADT (vpd)		FI Crashes (2014–2016)	Total Crashes (2014–2016)	Total Crash Rate per Million Entering Vehicles
	Inside Leg	Outside Leg	Exit	Entrance			
RD4ST	2,091	2,417	1,057	903	26	74	27.96
RA2ST	3,320	3,320	4,115	282	-	4	1.10
RB2ST	558	65	150	167	-	1	14.05
UD4ST	5,321	5,262	2,179	2,074	57	185	32.11
UA2ST	8,022	7,764	1,734	2,017	-	12	1.41
UB2ST	5,309	2,286	2,182	750	-	2	0.80
UD3enST	7,347	11,188	NA	3,533	-	8	0.65
UD3exST	8,726	8,226	3,674	NA	-	30	3.33
Total ST					**83**	**316**	**7.11**

Note: FI – Fatal and Injury, PDO – Property Damage Only, vpd – vehicles per day, Traveled, R – Rural, U – Urban, ST – Stop-Controlled, MEV – Million Entering Vehicles

Table 3.14 shows site characteristics and crash allocations of signal-controlled crossroad ramp terminals used for the calibration. There are 47 D4 signal-controlled crossroad ramp terminals and 27 other configurations of signal-controlled ramp terminals in this calibration sample. All signal-controlled crossroad ramp terminals considered were in urban settings. A majority of severe injury crashes occurred on D4 signal-controlled ramp terminals. Urban D4 signal-controlled crossroad ramp terminals reported the highest total crash rate per MEV.

Table 3.14: Crash Allocation and Site Characteristics of the Signal-Controlled Crossroad Ramp Terminal Sample

Ramp Facility (Signal-controlled Ramp Terminals)	Average Crossroad AADT (vpd)		Average Ramp AADT (vpd)		FI Crashes (2014–2016)	Total Crashes (2014–2016)	Total Crash Rate per Million Entering Vehicles
	Inside Leg	Outside Leg	Exit	Entrance			
UD4SG	12,354	12,448	4,477	4,024	138	542	39.76
UA2SG	12,962	12,416	4,458	2,664	11	51	3.75
UA4SG	11,282	19,150	8,217	2,125	5	8	0.38
UB2SG	13,242	13,448	3,816	3,710	31	62	4.21
UD3enSG	19,130	18,628	NA	4,211	4	11	0.54
UD3exSG	16,751	13,032	6,846	NA	9	49	3.43
Total SG					**198**	**723**	**7.55**

Note: FI – Fatal and Injury, PDO – Property Damage Only, vpd – vehicles per day, Traveled, R – Rural, U – Urban, SG – Signal-Controlled, MEV – Million Entering Vehicles

3.11 Aggregation of Databases to Perform Freeway Segment and Speed-Change Lane Calibration

Once the required data were gathered to perform the calibration, three main databases, as discussed in data preparation section, were merged in ArcGIS (Esri, 2012). Figure 3.31 indicates the merging procedure of three databases including actions taken to achieve the calibration. In this process, CANSYS database and ArcGIS shapefiles were utilized to obtain the required geometric and traffic data, and the KCARS database was used to extract reported crash data for the study period.

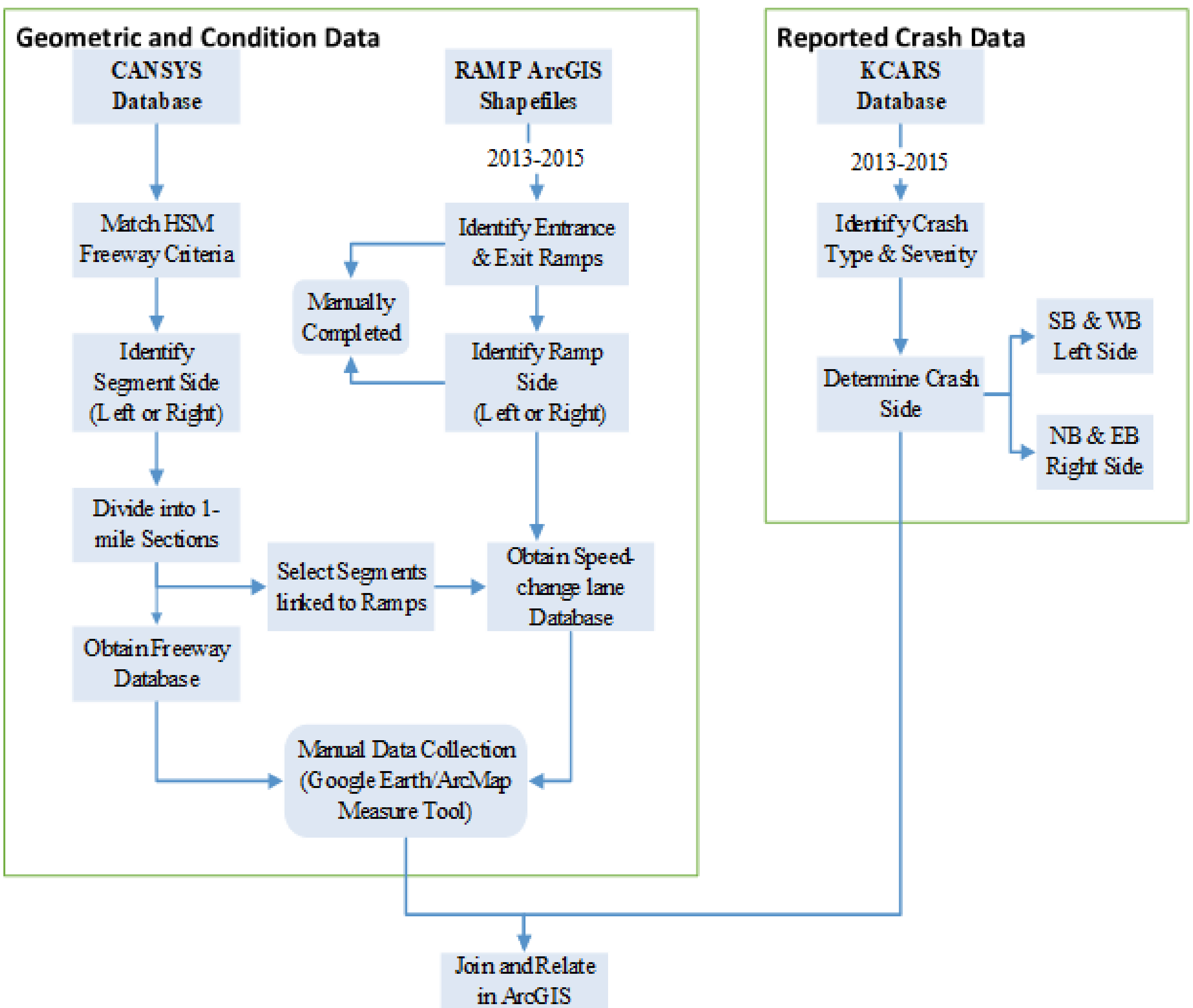


Figure 3.31: Aggregation of Databases in ArcGIS

3.12 Methodology

3.12.1 HSM Calibration Factor Estimation Process

This section summarizes the 18-step predictive method given in Figure 18-1 of the HSM supplement (AASHTO, 2014, pp. 18-8). Estimation of HSM calibration factors follows an ordinary process as shown in Figure 3.32. This is a simplified process from the 18-step predictive method.

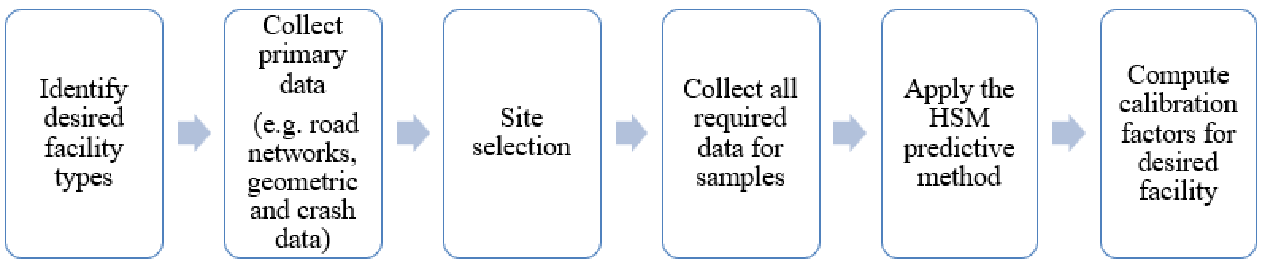


Figure 3.32: HSM Calibration Factor Development Process

3.12.2 HSM Predictive Method

The HSM predictive method follows a general form to compute the predicted average crash frequency as stated in Equation 3.3 (AASHTO, 2014). The model consists of a SPF developed for a set of base conditions, a set of CMFs to address non-base conditions, and a calibration factor.

$$\boldsymbol{N_{predicted} = N_{spf} \times (CMF_1 \times CMF_2 \times \ldots \times CMF_m) \times C} \qquad \textbf{Equation 3.3}$$

Where:

$N_{predicted}$ = Predicted average crash frequency,

N_{spf} = Safety performance function,

$CMF_{(1-m)}$ = Crash modification factors and,

C = Calibration factor.

Predictive models provided in Chapter 18 and Chapter 19 of the HSM supplement require separate analyses of segments and intersections. The HSM categorizes freeways and ramps as segments, and speed-change lanes and crossroad ramp terminals as intersections. The HSM predictive models for freeways, entrance speed-change lanes, and exit speed-change lanes apply to rural freeway segments with four to eight lanes and urban freeway segments with four to 10 lanes. The HSM predictive models for entrance and exit ramps apply to urban ramps with one to two lanes and rural ramps with one-lane. The HSM predictive models for signal-controlled and stop-controlled ramp terminals apply to rural crossroad ramp terminals with two to four crossroad lanes and urban crossroad ramp terminals with two to six crossroad lanes. Freeway and ramp crashes are predicted by combinations of two crash types and severity types such as multiple vehicle fatal and injury (MV FI), single vehicle fatal and injury (SV FI), multiple vehicle property damage only (MV PDO), and single vehicle property damage only (SV PDO). Speed-change lane crashes and crossroad ramp terminal crashes are predicted for all crash types and two crash severity types, FI and PDO.

3.12.3 Safety Performance Functions (SPFs)

The SPF predicts the number of crashes at a location as a function of the exposure and roadway characteristics. For freeway segments, speed-change lanes, and ramp segments, the exposure is signified by the segment length and AADT associated with the study section under a given set of base conditions. For crossroad ramp terminals, the exposure is signified by the AADTs associated with the crossroad, entrance ramps, and exit ramps. The calculation of the effective length of freeway segment (L*) was described in Section 3.5.1.4. Equation 3.4 denotes the SPF for multiple-vehicle or single-vehicle crashes on freeway segments.

$$\boldsymbol{N_{spf,freeways} = L^{*} \times exp\left(a + b \times ln\left[c \times AADT_{fs}\right]\right)}$$ **Equation 3.4**

Where:

$N_{spf,freeways}$ = Predicted average crash frequency,

L^{*} = Effective length of freeway segment (mi),

a, b = Regression coefficients,

c = AADT scale coefficient, and

$AADT_{fs}$ = AADT volume of freeway segment (vpd).

Equation 3.5 denotes the SPF for all crashes at entrance or exit speed-change lanes.

$$\boldsymbol{N_{spf,speed-change\ lanes}} = \boldsymbol{L_{en\ or\ ex}} \times \boldsymbol{exp\left(a + b \times ln\left[c \times AADT_{fs}\right]\right)}$$

Equation 3.5

Where:

$N_{spf,speed-change\ lanes}$ = Predicted average crash frequency,

$L_{en,ex}$ = length of ramp entrance or exit (mi),

a, b = Regression coefficients,

c = AADT scale coefficient, and

$AADT_{fs}$ = AADT volume of freeway segment (vpd).

Equation 3.6 denotes the SPF for multiple vehicle crashes on entrance or exit ramps and Equation 3.7 denotes the SPF for single vehicle crashes on entrance or exit ramps.

$$\mathbf{N_{spf,MV,ramps}} = \mathbf{L_r} \times \mathbf{exp(a + b \times [c \times AADT_r + d[c \times AADT_r])} \quad \textbf{Equation 3.6}$$

$$\mathbf{N_{spf,SV,ramps}} = \mathbf{L_r} \times \mathbf{exp(a + b \times [c \times AADT_r])} \quad \textbf{Equation 3.7}$$

Where:

$N_{spf,MV/SV,ramps}$ = Predicted average crash frequency,

L_r = length of ramp segment (mi),

a, b, d = Regression coefficients,

c = AADT scale coefficient, and

$AADT_r$ = AADT volume of ramp segment (vpd).

Equation 3.8 denotes the SPF for signal-controlled and stop-controlled crossroad ramp terminals.

$$\boldsymbol{N_{spf,SG/ST}} = \boldsymbol{exp(a + b \times ln[c \times AADT_{xrd}] + d \times ln[c \times AADT_{en} + c \times AADT_{ex}])}$$

with

$$\boldsymbol{AADT_{xrd}} = \mathbf{0.5} \times \boldsymbol{(AADT_{in} + AADT_{out})}$$

Equation 3.8

Where:

$N_{spf,SG/ST}$ = Predicted average crash frequency,

$AADT_{xrd}$ = AADT volume of the crossroad (vpd),

a, b, d = Regression coefficients,

c = AADT scale coefficient,

$AADT_{ex/en}$ = AADT volume of exit/entrance ramp (vpd),

$AADT_{in}$ = AADT volume of crossroad leg between ramps (vpd), and

$AADT_{out}$ = AADT volume of crossroad leg of interchange (vpd).

3.12.4 Crash Modification Factors (CMFs)

A CMF is a multiplicative factor used to measure the crash impact of geometric conditions. For example, if a segment has a lane width of 14 feet, which is a deviation from the base condition, the SPF can be modified by multiplying the relevant CMF associated with the lane width. For any segment characteristic that has counterparts to base conditions, the CMF value is equal to 1.0. A CMF value less than 1.0 indicates a reduction in predicted crashes while applying the specific countermeasure and vice versa. Table 3.15 provides CMFs that are applicable to freeway segments, speed-change lanes, ramp segments, and crossroad ramp terminals. In summary, a total of 11, 7, 8, 10, and 7 CMFs were calculated to perform the calibration for freeway segments, speed-change lanes, ramp segments, signal-controlled crossroad ramp terminals, and stop-controlled crossroad ramp terminals, respectively. The first six CMFs are common to both freeway segments and speed-change lanes. Several CMFs required a variable characterizing the proportion of the segment's length to a specific feature, such as a curve, barrier, or rumble strip, if it is present within the segment. In those circumstances, the proportion is equal to the total length of the feature summed over both roadbeds divided by the length of both roadbeds. Appendix C explains the estimation of the relevant CMFs for freeway segment calibration by providing a sample calculation.

Table 3.15: CMFs Applicable for Freeway Segments, Speed-Change Lanes, Ramp Segments, and Crossroad Ramp Terminals

CMF Variable	CMF Description	Freeway Segments	Entrance Speed-Change Lanes	Exit Speed-Change Lanes	Ramp Segments	Signal-Controlled Ramp Terminals	Stop-Controlled Ramp Terminals
CMF_1	Horizontal Curve	√	√	√	√		
CMF_2	Lane Width	√	√	√	√		
CMF_3	Inside Shoulder Width	√	√	√			
CMF_4	Median Width	√	√	√			
CMF_5	Median Barrier	√	√	√			
CMF_6	High Volume	√	√	√			
CMF_7	Lane Change	√					
CMF_8	Outside Shoulder Width	√					
CMF_9	Shoulder Rumble Strip	√					
CMF_{10}	Outside Clearance	√					
CMF_{11}	Outside Barrier	√					
CMF_{12}	Ramp Entrance		√				
CMF_{13}	Ramp Exit			√			
CMF_{14}	Right Shoulder Width				√		
CMF_{15}	Left Shoulder Width				√		
CMF_{16}	Right Side Barrier				√		
CMF_{17}	Left Side Barrier				√		
CMF_{18}	Lane Add or Drop				√		
CMF_{19}	Ramp Speed-Change Lane				√		
CMF_{20}	Weaving Section						
CMF_{21}	Exit Ramp Capacity					√	√
CMF_{22}	Crossroad Left-Turn Lane					√	√
CMF_{23}	Crossroad Right-Turn Lane					√	√
CMF_{24}	Access Point Frequency					√	√
CMF_{25}	Segment Length					√	√
CMF_{26}	Median Width					√	√
CMF_{27}	Protected Left-Turn Operation					√	
CMF_{28}	Channelized Right Turn on Crossroad					√	
CMF_{29}	Channelized Right Turn on Exit Ramp					√	
CMF_{30}	Non-Ramp Public Street Leg					√	
CMF_{31}	Skew Angle						√
Total CMFs		**11**	**7**	**7**	**8**	**10**	**7**

Source: AASHTO (2014)

3.12.5 Calibration Factor

According to the HSM, the calibration factor is computed by obtaining the ratio of total observed crashes to total predicted crashes. Observed crashes for freeway facilities were obtained from the KCARS database and observed crashes for ramp facilities were obtained by manually reviewing interchange-related crash reports. Predicted crashes were estimated using freeway and ramp facility SPFs from HSM and relevant CMFs. If the calibration factor is smaller than 1.00, the HSM methodology overpredicts crashes, and if the calibration factor is larger than 1.00, the HSM methodology underpredicts crashes. Overprediction designates a certain jurisdiction experiences less crashes than what is predicted using the HSM methodology and underprediction designates a certain jurisdiction experiences more crashes than what is predicted using the HSM methodology. Equation 3.9 was used to estimate the calibration factor.

$$\boldsymbol{C} = \frac{\sum_{i=1}^{n_{sites}} \sum_{j=1}^{n_{years}} \textbf{Observed crashes}}{\sum_{i=1}^{n_{sites}} \sum_{j=1}^{n_{years}} \textbf{Predicted crashes}} \qquad \textbf{Equation 3.9}$$

Where:

C = Calibration factor, and

n_{sites} = Number of sites of the freeway facility type

3.12.6 Development of Calibration Datasets

Calibration datasets were created in Microsoft Excel and geometric and traffic attributes of the sample segments or intersections were entered to the worksheet to perform calibrations. Then, CMFs were computed on different worksheets for each individual segment or intersection for cases where base conditions change. Next, the calibration procedure was executed on a new worksheet. The calibration worksheet included columns for CMFs, SPFs, predicted crash frequencies, observed crash frequencies, and estimated calibration factors as shown in Figure 3.24. The equations for SPFs were manually inserted and predicted crash frequencies were calculated for all freeway and ramp facility types considering each crash type and severity type combination for the entire study period. Then, the reported crashes for each individual segment were added to the calibration workbook. Finally, the calibration factors were estimated by dividing total observed crashes by total predicted crashes for the study period.

Freeway Segments in Kansas															
								MV							
ID	Prefix	Route ID	Number of Lanes	Segment Length (mi)	AADT fs (2015)	AADT fs (2014)	AADT fs (2013)	CMF 1 Horizontal Curve	CMF 2 Lane Width	CMF 3 Inside Shoulder Width	CMF 4 Median Width	CMF 5 Median Barrier	CMF 6 High Volume	CMF 7 Lane Change 2015	CMF 7 Lane Change 2014
1	I	056I0033500-NB	4LD - Four lane, divided	0.7600	7,890	7,320	6,550	1.0000	1.0000	0.9497	1.1490	1.0700	1.0000	1.0000	1.0000
2	I	056I0033500-NB	4LD - Four lane, divided	1.0000	7,890	7,320	6,550	1.0000	1.0000	0.9497	1.1490	1.0700	1.0000	1.0000	1.0000
3	I	056I0033500-NB	4LD - Four lane, divided	1.0000	7,890	7,320	6,550	1.0000	1.0000	0.9497	1.1490	1.0700	1.0000	1.0000	1.0000
4	I	056I0033500-NB	4LD - Four lane, divided	1.0000	7,890	7,320	6,550	1.0000	1.0000	0.9497	1.1490	1.0700	1.0000	1.0000	1.0000
5	U	061U0006900-NB	4LD - Four lane, divided	0.2966	9,270	9,230	8,130	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000

Crash Modification Factors																	
Fatal and Injury																	
	SV										MV						
CMF 7 Lane Change 2013	CMF 1 Horizontal Curve	CMF 2 Lane Width	CMF 3 Inside Shoulder Width	CMF 4 Median Width	CMF 5 Median Barrier	CMF 6 High Volume	CMF 8 Outside Shoulder Width	CMF 9 Shoulder Rumble Strips	CMF 10 Outside Clearance	CMF 11 Outside Barrier	CMF 1 Horizontal Curve	CMF 3 Inside Shoulder Width	CMF 4 Median Width	CMF 5 Median Barrier	CMF 6 High Volume	CMF 7 Lane Change 2015
1.0000	1.0000	1.0000	0.9497	0.9542	1.0700	1.0000	1.0000	0.8110	0.9592	1.0012	1.0000	0.9551	1.1432	1.0921	1.0000	1.0000
1.0000	1.0000	1.0000	0.9497	0.9542	1.0700	1.0000	1.0000	0.8110	0.9584	1.0009	1.0000	0.9551	1.1432	1.0921	1.0000	1.0000
1.0000	1.0000	1.0000	0.9497	0.9542	1.0700	1.0000	1.0000	0.8110	0.9609	1.0018	1.0000	0.9551	1.1432	1.0921	1.0000	1.0000
1.0000	1.0000	1.0000	0.9497	0.9542	1.0700	1.0000	1.0000	0.8110	0.9609	1.0018	1.0000	0.9551	1.1432	1.0921	1.0000	1.0000
1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	0.8110	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000

Property Damage Only									Safety Performance Functions							
		SV							2015				2014			
CMF 7 Lane Change 2014	CMF 7 Lane Change 2013	CMF 1 Horizontal Curve	CMF 3 Inside Shoulder Width	CMF 4 Median Width	CMF 5 Median Barrier	CMF 6 High Volume	CMF 8 Outside Shoulder Width	CMF 11 Outside Barrier	MV FI	SV FI	MV PDO	SV PDO	MV FI	SV FI	MV PDO	SV PDO
1.0000	1.0000	1.0000	0.9551	1.1422	1.0921	1.0000	1.0000	1.0015	0.0421	0.3444	0.0426	0.4966	0.0376	0.3281	0.0369	0.4650
1.0000	1.0000	1.0000	0.9551	1.1422	1.0921	1.0000	1.0000	1.0012	0.0554	0.4531	0.0561	0.6534	0.0495	0.4317	0.0485	0.6119
1.0000	1.0000	1.0000	0.9551	1.1422	1.0921	1.0000	1.0000	1.0023	0.0554	0.4531	0.0561	0.6534	0.0495	0.4317	0.0485	0.6119
1.0000	1.0000	1.0000	0.9551	1.1422	1.0921	1.0000	1.0000	1.0023	0.0554	0.4531	0.0561	0.6534	0.0495	0.4317	0.0485	0.6119
1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	0.0209	0.1491	0.0227	0.2232	0.0208	0.1487	0.0225	0.2223

				Predicted Crash Frequency											
2013				2015				2014				2013			
MV FI	SV FI	MV PDO	SV PDO	MV FI	SV FI	MV PDO	SV PDO	MV FI	SV FI	MV PDO	SV PDO	MV FI	SV FI	MV PDO	SV PDO
0.0319	0.3054	0.0297	0.4219	0.0492	0.2600	0.0508	0.5925	0.0440	0.2477	0.0440	0.5549	0.0372	0.2306	0.0354	0.5034
0.0420	0.4018	0.0391	0.5551	0.0647	0.3418	0.0669	0.7794	0.0578	0.3256	0.0578	0.7298	0.0490	0.3031	0.0466	0.6621
0.0420	0.4018	0.0391	0.5551	0.0647	0.3430	0.0669	0.7803	0.0578	0.3267	0.0578	0.7307	0.0490	0.3041	0.0466	0.6629
0.0420	0.4018	0.0391	0.5551	0.0647	0.3430	0.0669	0.7803	0.0578	0.3267	0.0578	0.7307	0.0490	0.3041	0.0466	0.6629
0.0172	0.1370	0.0176	0.1990	0.0209	0.1210	0.0227	0.2232	0.0208	0.1206	0.0225	0.2223	0.0172	0.1111	0.0176	0.1990

Observed Crash Frequency												Calibration Factors (2013-2015)			
2015				2014				2013							
MV FI	SV FI	MV PDO	SV PDO	MV FI	SV FI	MV PDO	SV PDO	MV FI	SV FI	MV PDO	SV PDO	MV FI	SV FI	MV PDO	SV PDO
0	0	0	3	0	0	0	2	0	0	1	2	0.9516	0.9364	1.9823	1.8432
0	0	0	1	0	0	0	0	0	0	0	3				
0	0	0	2	0	0	0	2	0	0	1	1				
0	0	0	2	0	0	0	1	0	0	0	1				
0	0	0	1	0	0	0	0	0	0	0	1				

Figure 3.33: Freeway Calibration Worksheet Sample

3.12.7 Assessment of the Quality of the Calibration Process

When calibration factors are developed for a certain facility type, jurisdictions must first assess the quality of the calibration process before developing local SPFs (Srinivasan et al., 2013). As the calibration factor is a single multiplier, it is recommended to evaluate the performance of the calibrated SPFs in explaining the inconsistency of reported crashes among sites (Lyon et al., 2016). Srinivasan et al. (2013) also listed a number of ways to assess the quality of the calibration process, as follows:

1. The value of the calibration factor – if the calibration factor is significantly different from 1.000 (i.e., much less or much greater), this indicates the crash experience in that specific jurisdiction is much different than the data used to estimate the initial SPFs.
2. Goodness-of-fit tests – cumulative residual (CURE) plots and coefficient of variation (CV), and the importance of CURE plots and CV are explained in the next sections. Further, the calibrated SPF is acceptable if either: (1) an upper threshold of 5% or less of CURE plot fitted (calibrated) values (after applying the calibration factor) exceed 2σ limits, or (2) the CV of the calibration factor is less than 0.15 (Hauer, 2016; Lyon et al., 2016).

3.12.7.1 Cumulative Residual (CURE) Plots

A CURE plot is a graphical indication of the cumulative residuals (observed minus predicted crashes) against the variable of interest arranged in ascending order. The variable of interest would be the calibration factors, AADT, segment length, shoulder width, etc. CURE plots provide further insight into whether the selected functional form is reasonable or not. In this scenario, CURE plots help to identify possible concerns by providing a visual depiction of goodness-of-fit over a range of variables of interest (Lyon et al., 2016). The concerns include:

1. Long trends: increasing or decreasing long trends in the CURE plot indicate bias in SPFs, therefore, jurisdictions must rectify improvement to the SPF by either changing the functional form or adding new variables.

2. Percent exceeding confidence limits: cumulative residuals outside the range of the two confidence intervals indicate a poor fit over the range of variable of interest.
3. Vertical changes: large vertical changes indicate potential outliers that may require further inspection.

Figure 3.34 graphically explains long trends, percent exceeding confidence limits, and vertical changes in a CURE plot. Fitted values in the CURE plots indicate the calibrated values, where the HSM predicted values are multiplied by the relevant calibration factor and cumulative residuals indicate the observed minus predicted crashes.

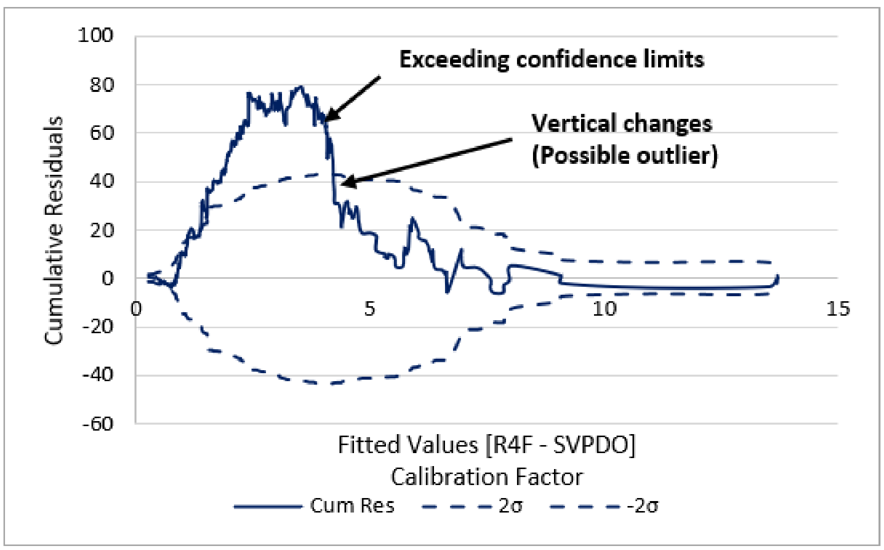


Figure 3.34: Possible concerns to be identified from CURE plots

3.12.7.2 Coefficient of Variation (CV) of the Calibration Factor

The CV of the calibration factor is the ratio of standard deviation of the calibration factor to the estimate of the calibration factor as shown in Equation 3.10. Variance of the calibration factor, V(C), is calculated using Equation 3.11. The square root of the variance is the standard deviation of the calibration factor.

$$CV = \frac{\sqrt{V(C)}}{C}$$ **Equation 3.10**

Where:

CV = Coefficient of variation of the calibration factor,

C = Estimate of the calibration factor, and

$V(C)$ = Variance of the calibration factor.

$$V(C) = \frac{\sum_{all\ sites}(y_i + k \times y_i^2)}{(\sum_{all\ sites} \hat{y})^2}$$ **Equation 3.11**

Where:

y_i = Observed counts,

$\hat{Y}$ = Uncalibrated predicted values from the SPF, and

k = Dispersion parameter (recalibrated).

The study used the Calibrator tool to develop two goodness-of-fit measures, the CURE plot and the CV, as discussed in this section (FHWA, n.d.). The spreadsheet-based Calibrator tool developed by Lyon et al. (2016) supports analysts in assessing SPF compatibility and applicability. The Calibrator tool computes several goodness-of-fit measures, such as mean absolute deviation, modified R^2, dispersion parameter, coefficient of variation (CV) of the calibration factor, and CURE plots, which could be used to determine if the SPF calibration is acceptable or not.

3.12.8 Development of Calibration Functions

Calibration functions are further developed to improve the fit of data compared to the use of calibration factors (Hauer, 2016; Srinivasan et al., 2016). A few recent studies related to HSM calibration mentioned that calibration functions deliver more accurate predictions than calibration factors and are more comparable with local SPFs (Claros et al., 2018; Farid, Abdel-Aty, & Lee, 2018; Rajabi, Ogle, & Gerard, 2018). It is therefore important to see whether the same is valid for conditions in Kansas. Equation 3.12 shows the base model of the calibration function, which signifies the relationship between observed crashes and predicted crashes in power function form commonly used in road safety modeling. If “b” is equal or close to the value of 1, “a” becomes the calibration factor. Apart from the base model, calibration functions could also be developed using other predictor variables, such as segment length, AADT, and

CMFs. However, this study was limited to the base calibration function. Common methods of estimating calibration functions are the ordinary least squares (OLS) method, Poisson regression, and NB regression. Among these three methods, Poisson regression is the most simple; however, NB regression is often used in the literature due to its capability to explain the overdispersion (Hauer, 2016; Srinivasan et al., 2016; Claros et al., 2018). The calibration functions were developed for freeway segments, speed-change lanes, and crossroad ramp terminals depending upon initial goodness-of-fit estimated using the two methods discussed in Section 3.12.7.

$$\boldsymbol{N_{obs}} = \boldsymbol{a} \times \boldsymbol{N_{pred}}^{\boldsymbol{b}}$$ **Equation 3.12**

Where:

N_{obs} = Observed crashes,

N_{pred} = Predicted crashes,

a = Model coefficient multiplier, and

b = Model coefficient exponent.

In the NB method, "a" and "b" parameters are estimated by maximizing the log-likelihood (LL). The LL based on the NB regression method was computed using Equation 3.13 (Hauer, 2016).

$$\ln[L(\boldsymbol{\beta_0}, \boldsymbol{\beta_1}, \dots, \beta)] = \sum_{i=1}^{n}[\boldsymbol{ln\Gamma}(\boldsymbol{obs_i} + \beta\boldsymbol{L_i}) - \boldsymbol{ln\Gamma}(\beta\boldsymbol{L_i}) + \beta\boldsymbol{L_i}\ln(\beta\boldsymbol{L_i}) + \boldsymbol{obs_i}\ln(\boldsymbol{pred_i}) - (\beta\boldsymbol{L_i} + \boldsymbol{obs_i}) \times \ln(\beta\boldsymbol{L_i} + \boldsymbol{pred_i})]$$

Equation 3.13

Where:

i = segment designation,

L_i = length of segment i,

$\beta_0, \beta_1, \dots, \beta$ = estimated parameters of model, and

$1/\beta L_i$ = dispersion parameter, "k."

All dispersion parameters in the HSM are either a constant or a function of segment length for road segments. The dispersion parameter "k" is determined from the variance equation as shown in Equation 3.14.

$$V = E + \frac{E^2}{6L_i}$$ **Equation 3.14**

Where:

V = estimated variance of mean crash rate,

E = estimated mean crash rate, and

$1/6L_i$ = the dispersion parameter, k.

3.12.9 The Calibrator

The Calibrator is a spreadsheet-based tool developed by FHWA that could be used to calibrate and assess the performance of SPFs (FHWA, n.d.). In addition, the latest version (2018) includes the capability of estimating calibration functions. The Calibrator tool creates CURE plots and provides goodness-of-fit measures to determine how well the function fits the data set. A more detailed procedure on use of the Calibrator tool is available in Lyon et al. (2016). This study used the Calibrator tool to assess the performance of calibrated SPFs and to estimate calibration functions; however, Microsoft Excel datasheets were developed to perform the calibration for all facilities.

This study estimated the calibration functions in two ways, using the NB method and the Calibrator tool (Hauer 2016; Srinivasan et al., 2016; FHWA, n.d.). The estimated values for "a" and "b" parameters were similar in both methods supporting that the Calibrator tool also uses the NB method in developing calibration functions.

Chapter 4: Results

The first part of this chapter provides estimated calibration factors, estimated goodness-of-fit measures, and developed calibration functions for all facility types considered in this study. The latter part of the chapter delivers developed Kansas-specific crash type distributions for freeway segments, speed-change lanes, and crossroad ramp terminals in comparison to HSM-default crash type distributions.

4.1 Estimated Calibration Factors for Freeway Segments and Speed-Change Lanes

Table 4.1 tabulates estimated calibration factors and developed calibration functions for freeway segments and speed-change lanes in Kansas. A calibration factor greater than 1.000 indicates an underprediction of crashes meaning that Kansas has experienced more crashes than what is predicted using the HSM predictive method. In contrast, a calibration factor smaller than 1.000 indicates an overprediction meaning that Kansas has experienced fewer crashes than what is predicted using the HSM predictive method. Considering all freeway segments, the HSM methodology overpredicted both MV FI and SV FI crashes, and underpredicted both MV PDO and SV PDO crashes. Considering all speed-change lane types, the HSM methodology consistently underpredicted both entrance- and exit- related FI and PDO crashes. Overall, the value of calibration factors for FI crashes was much closer to 1.000 compared to PDO crashes. When the calibration factors are significantly different from 1.000, it indicates some biasness in estimated calibration factors. This creates a necessity to further assess the quality of the calibration process as shown in previous studies related to the HSM calibration (Hauer, 2016; Lyon et al., 2016; Srinivasan et al., 2016; Srinivasan et al., 2013).

4.2 Assessment of Quality of Calibration Process for Freeway Facilities

The study estimated percent cure deviation and CV for freeway and speed-change lane SPFs as shown in Table 4.1. This assessment was only limited to the most-common types of Kansas freeways and the speed-change lanes in calibration sample, where the sample size was at least 50 sites for each facility type. It is acknowledged that smaller sample sizes may not result in reliable calibrations (Alluri et al., 2016; Bahar, 2014; Banihashemi, 2012; Kim et al., 2015; Trieu et al., 2014).

The percent of cure deviation for fitted values (after applying the calibration factor) less than 5%, or the CV of the calibration factor less than 0.15, indicates a successful calibration. The Calibrator tool was used to estimate the values for percent cure deviation and to create CURE plots (FHWA, n.d.). The percent cure deviation is the percentage of cumulative residuals lying outside the two standard deviations of the cumulative residuals (±2σ) in a CURE plot. In Figure 4.1, the x-axis represents fitted values and the y-axis cumulative residuals. The fitted values are the calibrated values where the HSM predicted crash frequency is multiplied by the estimated calibration factor. Cumulative residuals are calculated by subtracting the predicted crashes from the observed crashes.

Overall, considering either percent cure deviation or CV value for all freeway facility types considered in this study, it can be concluded that the calibration was reasonably successful. However, the percent of cure deviation for fitted values for the calibration factor was greater than 5% in majority of freeway models, thus indicating some biased predictions. Therefore, calibration functions were developed to further improve the fit of fitted values to the local dataset. Table 4.1 also provides "a" and "b" coefficients of developed calibration functions for disaggregate freeway and speed-change lane types. The quality assessment between calibrated HSM-default SPFs and developed calibration functions was conducted for freeway segment and speed-change lane SPFs using CURE plots. Furthermore, the estimated percent cure deviation values for calibration factors (i.e., calibrated HSM-default SPFs) and calibration functions are presented in Table 4.1.

Figure 4.1 displays CURE plots created for fitted values for calibration factor and calibration function for the rural 4-lane freeway, SV PDO model. For this specific scenario, the percent cure deviation was reduced from 60% to 6% when the calibration function was considered. Similarly, for all freeway facilities presented in Table 4.1, the percent cure deviation values (last two columns) reported for calibration functions are much less than the values reported for the respective calibration factors. This indicates that the developed calibration functions are better fitted for Kansas freeway segment and speed-change lane data compared to calibrated HSM-default SPFs.

Table 4.1: Estimated Calibration Factors, Estimated Goodness-of-Fit Measures, and Developed Calibration Functions for Freeway Segments and Speed-Change Lanes

Freeway Facility Type	Crash Type	Observed Crashes	Predicted Crashes	Calibration Factor (2013-2015)	Coefficient of Variation	Calibration Function		% Cure Deviation of Fitted Values	
						a	b	Calibration Factor	Calibration Function
R4F	MVFI	68	72.88	0.933	0.15	1.31	1.31	9	9
	SVFI	217	280.51	0.774	0.10	0.76	0.76	8	2
	MVPDO	220	99.92	2.202	0.14	2.24	1.10	11	9
	SVPDO	1,069	609.69	1.753	0.04	2.06	0.78	60	6
U4F	MVFI	83	89.63	0.926	0.18	0.92	0.94	1	1
	SVFI	150	123.46	1.215	0.14	1.20	0.50	27	5
	MVPDO	310	138.22	2.243	0.29	2.38	0.54	7	7
	SVPDO	662	313.48	2.112	0.07	2.98	0.63	79	1
All Freeways (HSM Criteria)	MVFI	219	230.13	0.952	0.14	0.96	1.06	7	6
	SVFI	441	470.95	0.936	0.09	0.95	0.76	49	1
	MVPDO	746	376.33	1.982	0.12	2.08	0.90	14	4
	SVPDO	2,046	1110.02	1.843	0.07	2.23	0.80	80	2
R4SC	ENFI	36	19.41	1.855	0.25	0.52	0.44	29	1
	ENPDO	130	66.13	1.966	0.15	1.44	0.70	1	0
	EXFI	33	26.38	1.251	0.26	2.06	1.25	8	0
	EXPDO	95	59.39	1.600	0.17	1.37	0.87	0	0
U4SC	ENFI	62	46.84	1.324	0.25	1.24	0.90	2	2
	ENPDO	180	103.72	1.736	0.16	1.79	0.78	16	6
	EXFI	30	32.06	0.936	0.34	0.98	1.03	6	6
	EXPDO	127	76.10	1.669	0.30	1.62	0.88	1	1
All Speed-Change Lanes (HSM Criteria)	ENFI	171	117.77	1.452	0.16	1.39	0.91	16	5
	ENPDO	567	291.83	1.943	0.13	1.95	0.99	6	6
	EXFI	124	87.60	1.416	0.20	1.70	1.20	18	0
	EXPDO	359	208.78	1.720	0.13	1.72	0.99	1	0

Note: MV – Multiple Vehicle, SV – Single Vehicle, FI – Fatal and Injury, PDO – Property Damage Only, EN – Entrance-related, EX – Exit-related

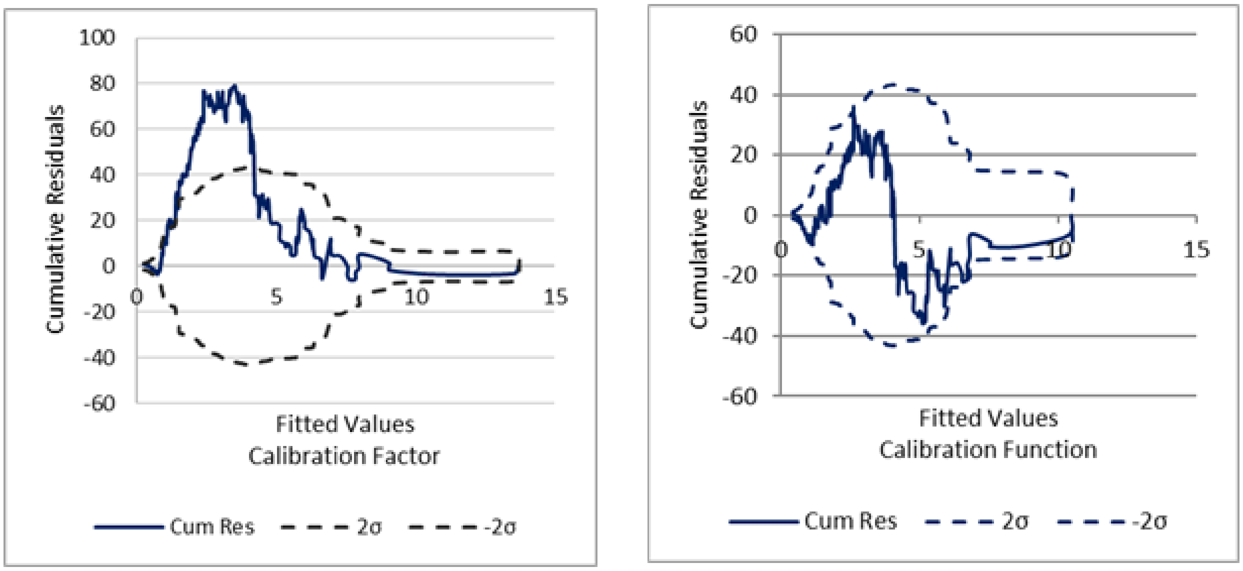


Figure 4.1: CURE Plots for Fitted Values for Calibration Factor and Calibration Function (Rural 4-Lane Freeways SV PDO Model)

4.3 Estimated Calibration Factors for Ramp Segments

Table 4.2 shows estimated calibration factors for ramp segments using Kansas data from 2014 to 2016. The HSM methodology overpredicted SV FI, SV PDO, and MV FI crashes and underpredicted MV PDO crashes for all entrance ramps in Kansas. In case of all exit ramps, the HSM methodology underpredicted all MV crashes and overpredicted all SV crashes. Moreover, results indicated MV ramp crashes are more prevalent in urban settings then in rural settings in Kansas. When no crashes were reported during the study period for a particular ramp type, calibration factors for those ramp types were considered as 1.000. Calibration factors were also estimated separately for urban and rural settings. However, none of the prediction models met the HSM crash criteria of 100 crashes per year even with a satisfactory sample size of 184 and 156, respectively, for entrance and exit ramps. Furthermore, sample sizes of 184 entrance ramps and 156 exit ramps considered in this study were much larger than the HSM-desired sample size requirement of 30 to 50 sites. Additionally, rural ramps experienced very few total crashes in comparison to urban ramps. Calibration factors for multiple vehicle crashes at exit ramps were much greater than 1.000 compared to calibration factors for multiple vehicle crashes at entrance ramps.

Nevertheless, assessment of calibrated HSM-default SPFs for ramp segments was not conducted in this study. The HSM ramp SPFs are to be considered by number of lanes, area type, and severity type, and therefore, the number of crashes relevant to each SPF becomes small. Consequently, it is not practicable to assess the performance of these calibrated HSM-default ramp SPFs or to develop calibration functions having smaller crash numbers, since it is well known that smaller crash numbers do not provide reliable results.

Table 4.2: Estimated Calibration Factors for Kansas Ramp Segments

Ramp Facility	Crash Type	No. of Segments	Observed Crashes	Predicted Crashes	Calibration Factor (2014–2016)
Urban EN	MVFI	122	10	10.28	0.973
	SVFI	122	6	38.66	0.155
	MVPDO	122	52	18.28	2.845
	SVPDO	122	17	52.21	0.326
Rural EN	MVFI*	62	0	0.17	1.000
	SVFI	62	1	3.86	0.259
	MVPDO*	62	0	0.72	1.000
	SVPDO	62	4	4.80	0.833
All EN Ramps (HSM Criteria)	MVFI	184	10	10.45	0.957
	SVFI	184	7	42.52	0.165
	MVPDO	184	52	19.00	2.737
	SVPDO	184	21	57.01	0.368
Urban EX	MVFI	96	7	1.26	5.556
	SVFI	96	5	24.59	0.203
	MVPDO	96	35	4.18	8.373
	SVPDO	96	20	37.43	0.534
Rural EX	MVFI*	60	0	0.03	1.000
	SVFI*	60	0	3.29	1.000
	MVPDO*	60	0	0.21	1.000
	SVPDO	60	3	4.40	0.682
All EX Ramps (HSM Criteria)	MVFI	156	7	1.29	5.426
	SVFI	156	5	27.88	0.179
	MVPDO	156	35	4.39	7.973
	SVPDO	156	23	41.84	0.550

Note: * denotes that no crashes were reported during the study period and calibration factor was considered as 1.000. MV – Multiple Vehicle, SV – Single Vehicle, FI – Fatal and Injury, PDO – Property Damage Only

4.4 Estimated Calibration Factors for Crossroad Ramp Terminals

Table 4.3 provides estimated calibration factors for all stop-controlled and signal-controlled crossroad ramp terminals in Kansas. For the calibration, 120 stop-controlled crossroad ramp terminals and 74 signal-controlled ramp terminals were used. The HSM-classified crossroad ramp terminal configuration types included in the calibration are D4, A2, B2, A4, D3en, and D3ex. The number of sites belonging to each crossroad terminal configuration is provided in Table 3.8. Calibration factors were also developed by area type for all stop-controlled and signal-controlled crossroad ramp terminals as shown in Table 4.3, although all signal-controlled crossroad ramp terminals are located in urban settings. Considering all stop-controlled and signal-controlled crossroad ramp terminals, the HSM methodology underpredicted FI crashes and overpredicted PDO crashes.

Table 4.4 shows estimated calibration factors for D4 stop-controlled and D4 signal-controlled crossroad ramp terminals in Kansas. Calibration factors for D4 crossroad ramp terminals were estimated separately because D4 is the most common ramp terminal configuration type present in Kansas. For the calibration, 102 D4 stop-controlled crossroad ramp terminals and 47 D4 signal-controlled crossroad ramp terminals were used. The HSM methodology underpredicted both FI and PDO crashes at D4 stop-controlled crossroad ramp terminals. Considering D4 signal-controlled crossroad ramp terminals, the HSM methodology overpredicted FI crashes and underpredicted PDO crashes.

4.5 Assessment of Quality of Calibration Process for Crossroad Ramp Terminals

Considering either percent cure deviation or CV value for both signal-controlled and stop-controlled crossroad ramp terminals as shown in Table 4.3 and Table 4.4, the calibration is acceptable with some biased predictions. However, considering only the percent of cure deviation for fitted values for the calibration factor, values were greater than 5% in a majority of cases, and therefore, calibration functions were developed to further improve the fit to local data. Table 4.3 and Table 4.4 also provide estimated “a” and “b” coefficients, and respective percent cure deviations for calibration functions. Developed calibration functions improved the fit to local data, resulting in lower percent cure deviations compared to those of calibration factors.

Table 4.3: Estimated Calibration Factors, Estimated Goodness-of-Fit Measures, and Developed Calibration Functions for All Stop-Controlled and Signal-Controlled Crossroad Ramp Terminals

Ramp Terminal Facility (All)	Crash Type	Observed Crashes	Predicted Crashes	Calibration Factor (2014–2016)	Coefficient of Variation	Calibration Function		% Cure Deviation of Fitted Values	
						a	b	Calibration Factor	Calibration Function
Rural ST	STFI	26	25.71	1.011	0.21	0.92	0.55	49	6
	STPDO	53	35.75	1.483	0.15	1.46	0.42	65	0
Urban ST	STFI	57	68.15	0.836	0.38	0.85	0.91	2	3
	STPDO	180	136.44	1.319	0.12	1.66	0.72	15	7
All ST (HSM Criteria)	STFI	83	93.86	0.884	0.27	0.91	0.72	36	8
	STPDO	233	172.19	1.353	0.10	1.70	0.65	49	3
All SG (Urban) (HSM Criteria)	SGFI	198	316.45	0.626	0.17	1.51	0.43	53	1
	SGPDO	525	422.75	1.242	0.12	3.34	0.46	77	5

Note: FI – Fatal and Injury, PDO – Property Damage Only, ST – Stop-Controlled, SG – Signal-Controlled

Table 4.4: Estimated Calibration Factors, Estimated Goodness-of-Fit Measures, and Developed Calibration Functions for D4 Stop-Controlled and Signal-Controlled Crossroad Ramp Terminals

Ramp Terminal Facility (D4)	Crash Type	Observed Crashes	Predicted Crashes	Calibration Factor (2014–2016)	Coefficient of Variation	Calibration Function		% Cure Deviation of Fitted Values	
						a	b	Calibration Factor	Calibration Function
Rural ST	STFI	26	25.16	1.033	0.20	0.95	0.55	51	6
	STPDO	48	35.23	1.362	0.15	1.37	0.47	57	2
Urban ST	STFI	57	49.05	1.162	0.28	1.14	1.10	2	0
	STPDO	128	103.50	1.237	0.14	1.64	0.64	9	5
All ST (HSM Criteria)	STFI	83	74.21	1.118	0.22	1.14	0.76	26	9
	STPDO	176	138.73	1.269	0.10	1.61	0.59	45	8
All SG (Urban) (HSM Criteria)	SGFI	138	205.54	0.671	0.20	1.61	0.46	34	6
	SGPDO	404	266.66	1.515	0.12	3.38	0.57	32	2

Note: FI – Fatal and Injury, PDO – Property Damage Only, ST – Stop-Controlled, SG – Signal-Controlled

4.6 Crash Type Distributions for Freeway Segments, Speed-Change Lanes, and Crossroad Ramp Terminals in Kansas

The HSM provides a default distribution of crash type for each predictive model in Chapter 18 and Chapter 19 (AASHTO, 2014, pp.18-13, pp.19-15). Development of crash type distributions for freeway and ramp facilities is extremely useful for jurisdictions because they can be used to estimate the expected average crash frequency for each 10 crash types (e.g., head-on, crash with animal). Therefore, it is beneficial to update the HSM-default crash type distributions with locally derived values as a part of the calibration process. However, this could be only developed if there are adequate numbers of reported crashes for each facility as defined in the HSM supplement (AASHTO, 2014, pp. Appendix B-10).

Table 4.5 through Table 4.9 provide developed Kansas-specific crash type distributions in comparison to HSM-default crash type distributions for freeway segments, speed-change lanes, and crossroad ramp terminals, respectively. Development of Kansas-specific crash type distributions for ramp segments was not possible because ramp segments considered in this study did not meet the required total crash frequency of 200 crashes for developing jurisdiction-specific crash type distributions. In addition, Kansas does not specifically record “right-angle” crashes as one of the MV crash types as defined in Chapter 18 and Chapter 19 of the HSM. However, Kansas does record all “angle crashes”—which include right-angle and other angle crashes. Table 4.5 provides developed Kansas-specific crash type distributions for freeway segments using crash data from 2013 to 2015.

Table 4.5: Kansas Crash Distributions for Freeway Segments

Area Type	Crash Type	Crash Type Category	HSM-Default		Kansas-Specific	
			FI	PDO	FI	PDO
Rural	Multiple Vehicle	Head-on	0.018	0.004	0.006	0.007
		Right-angle*	0.056	0.030	0.254	0.150
		Rear-end	0.630	0.508	0.581	0.428
		Sideswipe	0.237	0.380	0.146	0.375
		Other MV crashes	0.059	0.078	0.014	0.041
	Single Vehicle	Crash with animal	0.010	0.065	0.086	0.471
		Crash with fixed object	0.567	0.625	0.493	0.346
		Crash with other object	0.031	0.125	0.016	0.043
		Crash with parked vehicle	0.024	0.023	0.024	0.014
		Other SV crashes	0.368	0.162	0.381	0.125
Urban	Multiple Vehicle	Head-on	0.008	0.002	0.023	0.005
		Right-angle*	0.031	0.018	0.072	0.081
		Rear-end	0.750	0.690	0.695	0.557
		Sideswipe	0.180	0.266	0.201	0.323
		Other MV crashes	0.031	0.024	0.008	0.034
	Single Vehicle	Crash with animal	0.004	0.022	0.026	0.155
		Crash with fixed object	0.722	0.716	0.814	0.656
		Crash with other object	0.051	0.139	0.020	0.074
		Crash with parked vehicle	0.015	0.016	0.022	0.017
		Other SV crashes	0.208	0.107	0.117	0.098

Note: *Kansas records all angle-crashes

Kansas freeway segment crash proportions for both FI and PDO crashes were higher compared to HSM-default crash type distributions for MV "right-angle" crash types in both rural and urban settings, because MV crashes by "right-angle" crash type are not directly reported in Kansas. In addition, "rear-end" and "sideswipe" crash types recorded comparatively lower FI and PDO crash proportions compared to HSM-default values for both urban and rural settings. Considering freeway SV crashes in Kansas, both FI and PDO crash proportions were higher for the "collision with animal" crash type compared to relevant HSM-default crash proportions. These values seem to be reliable because animal or deer crashes in Kansas are higher compared to the states of Washington, Maine, and California (KDOT, 2018). Similarly, Table 4.6 and Table 4.7 provide Kansas-specific crash type distributions developed for entrance- and exit-related speed-change lanes using crash data from 2013 to 2015, respectively.

Table 4.6: Kansas Crash Distributions for Entrance-Related Speed-Change Lanes

Area Type	Crash Type	Crash Category	HSM-Default		Kansas-Specific	
			FI	PDO	FI	PDO
Rural	Multiple Vehicle	Head-on	0.021	0.004	0.000	0.000
		Right-angle*	0.032	0.013	0.087	0.047
		Rear-end	0.351	0.260	0.130	0.094
		Sideswipe	0.128	0.242	0.043	0.156
		Other multiple vehicle crashes	0.011	0.040	0.000	0.000
	Single Vehicle	Crash with animal	0.000	0.009	0.174	0.266
		Crash with fixed object	0.245	0.296	0.261	0.344
		Crash with other object	0.021	0.070	0.043	0.031
		Crash with parked vehicle	0.021	0.000	0.000	0.016
		Other single vehicle crashes	0.170	0.066	0.261	0.047
Urban	Multiple Vehicle	Head-on	0.004	0.001	0.020	0.007
		Right-angle*	0.019	0.016	0.040	0.041
		Rear-end	0.543	0.530	0.420	0.324
		Sideswipe	0.133	0.252	0.140	0.209
		Other multiple vehicle crashes	0.017	0.015	0.000	0.027
	Single Vehicle	Crash with animal	0.000	0.002	0.020	0.081
		Crash with fixed object	0.194	0.129	0.180	0.182
		Crash with other object	0.019	0.036	0.000	0.061
		Crash with parked vehicle	0.004	0.003	0.000	0.000
		Other single vehicle crashes	0.067	0.016	0.180	0.068

Note: *Kansas records all angle-crashes

Similar to freeway segments, developed crash proportions for "right-angle" crashes recorded higher values in Kansas as they include all "angle crashes." Overall, developed crash proportions for "rear-end" and "sideswipe" crash types are relatively lower than the HSM-default values for both entrance- and exit-related speed change lanes. Once again, Kansas reported higher crash type proportions for "crash with animal" in both entrance- and exit-related speed-change lanes.

Table 4.7: Kansas Crash Distributions for Exit-Related Speed-Change Lanes

Area Type	Crash Type	Crash Category	HSM-Default		Kansas-Specific	
			FI	PDO	FI	PDO
Rural	Multiple Vehicle	Head-on	0.000	0.000	0.000	0.000
		Right-angle*	0.015	0.000	0.000	0.000
		Rear-end	0.463	0.304	0.421	0.032
		Sideswipe	0.104	0.243	0.105	0.111
		Other multiple vehicle crashes	0.000	0.009	0.053	0.000
	Single Vehicle	Crash with animal	0.000	0.061	0.000	0.270
		Crash with fixed object	0.224	0.235	0.263	0.349
		Crash with other object	0.030	0.061	0.000	0.048
		Crash with parked vehicle	0.000	0.017	0.053	0.016
		Other single vehicle crashes	0.164	0.070	0.105	0.175
Urban	Multiple Vehicle	Head-on	0.005	0.002	0.045	0.006
		Right-angle*	0.011	0.012	0.091	0.017
		Rear-end	0.549	0.565	0.386	0.436
		Sideswipe	0.158	0.138	0.045	0.099
		Other multiple vehicle crashes	0.016	0.016	0.023	0.017
	Single Vehicle	Crash with animal	0.000	0.007	0.000	0.093
		Crash with fixed object	0.196	0.207	0.205	0.267
		Crash with other object	0.016	0.030	0.023	0.029
		Crash with parked vehicle	0.000	0.000	0.000	0.000
		Other single vehicle crashes	0.049	0.023	0.182	0.035

Note: *Kansas records all angle-crashes

Table 4.8 shows Kansas-specific crash type distributions developed for stop-controlled crossroad ramp terminals using crash data from 2014 to 2016. All locally derived MV and SV crash type distributions for stop-controlled crossroad ramp terminals were much closer to HSM-default crash type distributions for both rural and urban settings. Like other facility types, the local derived “right-angle” crash proportion includes all angle crashes. In addition, the SV “crash with fixed object” crash proportion was higher compared to the HSM-default value.

Table 4.8: Kansas Crash Distributions for Stop-Controlled Crossroad Ramp Terminals

Area Type	Crash Type	Crash Category	HSM-Default		Kansas-Specific	
			FI	PDO	FI	PDO
Rural	Multiple Vehicle	Head-on	0.020	0.015	0.063	0.000
		Right-angle*	0.522	0.372	0.531	0.281
		Rear-end	0.275	0.276	0.156	0.316
		Sideswipe	0.020	0.107	0.063	0.088
		Other multiple vehicle crashes	0.013	0.026	0.031	0.018
	Single Vehicle	Crash with animal	0.000	0.000	0.000	0.000
		Crash with fixed object	0.078	0.158	0.094	0.193
		Crash with other object	0.000	0.005	0.000	0.000
		Crash with parked vehicle	0.007	0.015	0.000	0.000
		Other single vehicle crashes	0.065	0.026	0.063	0.105
Urban	Multiple Vehicle	Head-on	0.017	0.012	0.105	0.020
		Right-angle*	0.458	0.378	0.421	0.337
		Rear-end	0.373	0.377	0.368	0.455
		Sideswipe	0.025	0.079	0.026	0.119
		Other multiple vehicle crashes	0.017	0.016	0.000	0.000
	Single Vehicle	Crash with animal	0.000	0.000	0.000	0.020
		Crash with fixed object	0.085	0.110	0.000	0.040
		Crash with other object	0.000	0.000	0.000	0.000
		Crash with parked vehicle	0.000	0.008	0.000	0.000
		Other single vehicle crashes	0.025	0.020	0.079	0.010

Note: *Kansas records all angle-crashes

Table 4.9 tabulates Kansas-specific crash type distributions developed for signal-controlled crossroad ramp terminals using crash data from 2014 to 2016. All signal-controlled crossroad ramp terminal crashes considered in this study were limited to urban settings. Once again, "right-angle" crash proportions for both FI and PDO crashes were higher in Kansas compared to HSM-default crash type proportions, mainly because locally derived values include all "angle crashes." Kansas-specific crash type proportion for FI "head-on" crash type was higher compared to the HSM-default value.

Table 4.9: Kansas Crash Distribution for Signal-Controlled Crossroad Ramp Terminals

Area Type	Crash Type	Crash Category	HSM-Default		Kansas-Specific	
			FI	PDO	FI	PDO
Urban	Multiple Vehicle	Head-on	0.011	0.007	0.035	0.005
		Right-angle*	0.260	0.220	0.509	0.425
		Rear-end	0.625	0.543	0.386	0.482
		Sideswipe	0.042	0.149	0.018	0.057
		Other multiple vehicle crashes	0.009	0.020	0.000	0.005
	Single Vehicle	Crash with animal	0.000	0.000	0.000	0.000
		Crash with fixed object	0.033	0.050	0.018	0.016
		Crash with other object	0.001	0.002	0.018	0.005
		Crash with parked vehicle	0.001	0.002	0.000	0.000
		Other single vehicle crashes	0.018	0.007	0.018	0.005

Note: *Kansas records all angle-crashes

Developed Kansas-specific crash type distributions for freeway segments, speed-change lanes, and crossroad ramp terminals in this section can replace the HSM-default crash type distributions to obtain more accurate crash predictions by crash type in Kansas. Once the expected average crash frequency for a certain segment or intersection is estimated, it is then multiplied by the developed Kansas-specific crash type proportion to compute the expected average crash frequency for each of 10 crash types.

Chapter 5: Summary, Conclusions, and Recommendations

5.1 Research Summary

The HSM is a national handbook that provides a quantitative evaluation of safety in highway facilities. When applying HSM prediction models to a local jurisdiction, they need to be calibrated to reflect local conditions and environments such as weather conditions, animal population, topography, crash-reporting thresholds, highway conditions, driving culture, and lighting. Considering Kansas freeway crash data from 2012 to 2016, freeways had the highest FI and PDO crash rates per mile compared to other functional classes. As KDOT maintains more than 1,000 miles of freeways, it is beneficial to have accurate crash prediction models for Kansas freeway and ramp facilities, which could be used in making effective planning, design, operation, and maintenance decisions. As freeway and ramp facility SPFs and their calibration procedures are comparatively new, not many studies have been conducted under this directive.

This study calibrated HSM SPFs for freeway segments, speed-change lanes, ramp segments, and crossroad ramp terminals for Kansas conditions. The freeway segment and speed-change lane calibration used three years of crash data from 2013 to 2015, with freeway crash data extracted from the KCARS database. The study period for the ramp segment and crossroad ramp terminal calibration was considered as 2014 to 2016; however, the study manually reviewed 13,730 interchange-related crash reports to identify crashes on ramp facilities. This study assessed the accuracy of calibrated HSM-default SPFs using CURE plots and CV values. For comparison purposes and further accuracy, calibration functions were developed to improve fit to local data. Table 5.1 summarizes the estimated calibration factors and developed calibration functions with respect to the HSM-recommended prediction models that need calibration for all freeway and ramp facilities considered in this study. The last column in Table 5.1 indicates an "overprediction," if the estimated calibration factor is less than 1.000, and an "underprediction," if the estimated calibration factor is greater than 1.000.

Table 5.1: Summary of HSM Calibration Results of this Study

Facility Type	Crash Type	Number of Sites Used	Observed Crashes	Predicted Crashes	Calibration Factor	Overprediction/ Underprediction	Calibration Function	
							a	b
Freeway Segments	MVFI	521	219	230.13	0.952	Overprediction	0.96	1.06
	SVFI	521	441	470.95	0.936	Overprediction	0.95	0.76
	MVPDO	521	746	376.33	1.982	Underprediction	2.08	0.90
	SVPDO	521	2,046	1,110.02	1.843	Underprediction	2.23	0.80
Speed-Change Lanes	ENFI	351	171	117.77	1.452	Underprediction	1.39	0.91
	ENPDO	351	567	291.83	1.943	Underprediction	1.95	0.99
	EXFI	366	124	87.60	1.416	Underprediction	1.70	1.20
	EXPDO	366	359	208.78	1.720	Underprediction	1.72	0.99
Entrance Ramp Segments	MVFI	184	10	10.45	0.957	Overprediction	NA	NA
	SVFI	184	7	42.52	0.165	Overprediction	NA	NA
	MVPDO	184	52	19.00	2.737	Underprediction	NA	NA
	SVPDO	184	21	57.01	0.368	Overprediction	NA	NA
Exit Ramp Segments	MVFI	156	7	1.29	5.426	Underprediction	NA	NA
	SVFI	156	5	27.88	0.179	Overprediction	NA	NA
	MVPDO	156	35	4.39	7.973	Underprediction	NA	NA
	SVPDO	156	23	41.84	0.550	Overprediction	NA	NA
Stop-Controlled Ramp Terminals	STFI	120	83	93.86	0.884	Overprediction	0.91	0.72
	STPDO	120	233	172.19	1.353	Underprediction	1.70	0.65
Signal-Controlled Ramp Terminals*	SGFI	74	198	316.45	0.626	Overprediction	1.51	0.43
	SGPDO	74	525	422.75	1.242	Underprediction	3.34	0.46
D4 Stop-Controlled Ramp Terminals	STFI	102	83	74.21	1.118	Underprediction	1.14	0.76
	STPDO	102	176	138.73	1.269	Underprediction	1.61	0.59
D4 Signal-Controlled Ramp Terminals*	SGFI	57	138	205.54	0.671	Overprediction	1.61	0.46
	SGPDO	57	404	266.66	1.515	Underprediction	3.38	0.57

Note: *includes only urban sites
MV – Multiple Vehicle, SV – Single Vehicle, FI – Fatal and Injury, PDO – Property Damage Only, EN – Entrance-related, EX – Exit-related, ST – Stop-Controlled, SG – Signal-Controlled, NA – Calibration functions were not developed

5.2 Conclusions

Estimated calibration factors for freeway segments showed the HSM methodology overpredicted all FI crashes and underpredicted all PDO crashes. The HSM methodology consistently underpredicted both FI and PDO crashes for entrance and exit speed-change lanes. The quality assessment of calibrated HSM-default SPFs was satisfactory for both freeway segments and speed-change lanes considering either the CV or percent cure deviation; however, for comparison purposes and further accuracy, calibration functions were developed. This study did not assess the quality of calibrated HSM-default SPFs or develop calibration functions for freeway facility types, such as rural 6-lane, urban 6-lane, and urban 8-lane freeways due to smaller sample sizes. Then estimated values for percent cure deviation were compared between developed calibration functions and estimated calibration factors. Results showed that developed calibration functions for freeway segments and speed-change lanes fitted better to local data compared to estimated calibration factors, which reported lower percent cure deviation values.

The HSM methodology overpredicted all FI crashes, underpredicted MV PDO crashes, and overpredicted SV PDO crashes for entrance ramp segments. The HSM methodology underpredicted all MV crashes and overpredicted all SV crashes for exit ramp segments. The quality assessment of calibrated HSM-default ramp SPFs and the development of calibration functions for entrance and exit ramps were not conducted due to a lower number of observed crashes reported during the study period. In the case of stop- and signal-controlled crossroad ramp terminals, the HSM methodology overpredicted all FI crashes and underpredicted all PDO crashes. The study also estimated calibration factors separately for D4 crossroad ramp terminals as these are the most common ramp terminal configuration type in Kansas. The HSM methodology underpredicted both FI and PDO crashes for D4 stop-controlled crossroad ramp terminals. In the case of D4 signal-controlled crossroad ramp terminals, the HSM methodology underpredicted PDO crashes and overpredicted FI crashes. Even though the quality of calibrated HSM-default SPFs was satisfactory, calibration functions were also developed for all crossroad ramp terminals and D4 types for comparison purposes and further accuracy. Similar to calibration functions developed for freeway facility types considered, calibration functions developed for crossroad ramp terminals fitted better to the local dataset compared to estimated calibration factors. Estimated calibration factors for PDO crash models were much higher compared to FI crash models for all freeway and ramp facility types considered in this study. One valid explanation could be that PDO crashes are accurately reported in Kansas compared to the states of California, Washington, and Maine that were used to develop HSM crash prediction models for freeway and ramp facilities.

5.3 Challenges Faced and Areas for Improvement

This section provides challenges faced in conducting this calibration study together with explanations on addressing those listed challenges. These challenges and explanations would benefit and encourage researchers to conduct other similar calibrations. Table 5.2 summarizes challenges faced and how they were addressed as this study progressed. Compared to the calibration of freeway segments and speed-change lanes, calibration of ramp segments and crossroad ramp terminals required much more extensive data, where most transportation agencies do not record such detailed geometric data for ramp facility types in typical motor vehicle crash reports. In the case of freeway segments and speed-change lanes, a majority of required geometric, traffic, and crash data needed for the calibration were available in electronic data sources maintained by KDOT. However, for ramp segments and crossroad ramp terminals, an overwhelming majority of geometric data needed for the calibration was extracted from Google Earth, a highly time-consuming task.

Even though all freeway crashes were available for the desired study period, locating crashes on entrance- and exit-related speed-change lanes was a significant challenge. In addition, accurately locating crashes on ramps and crossroads ramp terminals was the most critical challenge. While all ramp crashes and ramp terminal crashes were included in the KCARS database under interchange-related crashes, they could not be directly extracted with respect to the exact crash location. Therefore, it was necessary to manually review all interchange-related crash reports from 2014 to 2016 to identify crashes that occurred on ramps and crossroad ramp terminals. After manually reviewing all interchange-related crash reports from several counties in Kansas, it was identified that 90% of crashes coded as ramp crashes in the KCARS database were actually located on crossroad ramp terminals or speed-change lanes.

Table 5.2: Challenges Faced and Tips to Succeed in HSM Freeway Calibration

Challenges	How challenges were addressed
Locating speed-change lane crashes – Crashes for entrance and exit speed-change lanes could not be directly identified from the KCARS database.	All freeway crashes were assigned to each side of the freeway and ArcGIS tools were used to isolate speed-change lane crashes as described in Section 3.7.3.
Locating ramp segment crashes/Discrepancy between actual and coded crashes in KCARS database – Most of the multiple vehicle crashes that were identified as ramp crashes in the KCARS database were actually located on ramp terminals or speed-change lanes.	This study manually reviewed full crash reports for all interchange-related crashes during the study period to identify ramp crashes as explained in Section 3.9.3.
Locating crossroad ramp terminal crashes – Crossroad ramp terminal crashes could not be directly identified from the KCARS database.	While identifying all ramp crashes, crashes for stop-controlled and signal-controlled crossroad ramp terminals were also identified.
Meeting the HSM minimum crash criteria for ramp segments and stop-controlled crossroad ramp terminals – The HSM recommends a desirable sample of 30 to 50 sites with having at least 100 crashes per year for the calibration.	The study proceeded the calibration with the actual reported crash numbers, even though they did not satisfy the HSM requirements.
None of the required geometric data for the ramp segment and crossroad ramp terminal calibration were readily available in Kansas electronic databases.	Most of the geometric data required for the calibration were gathered using Google Earth.
Ramp segmentation based on number of lanes – Ramp segmentation by number of lanes was not considered due to the inability to locate crashes at the exact location on ramps.	In this study, if the ramp has three quarter of its length as 2-lane this ramp was considered as a 2-lane ramp as explained in Section 3.7.4.
Obtaining ramp horizontal curve radius – The horizontal curve radii for ramps were not readily available in Kansas electronic databases. The study first imported Google Earth images to AutoCAD to obtain curve radii of ramps. However, it was highly dependent on personal judgement and it was not easy to identify the beginning of the curve and end of the curve.	KDOT uses a build-in ArcGIS curve tool to obtain the curve radius in roadways. This tool was applied to the latest ramp GIS shapefile to obtain horizontal curve radii for ramps in Kansas.

Moreover, obtaining horizontal curve radii for ramp segments and ramp segmentation by number of lanes was also a huge challenge. Fulfilling the HSM-desired crash criteria for ramp segments and stop-controlled crossroad ramp terminals were also huge challenges. Even though significantly larger sample sizes were considered for both of these facility types, meeting 300 crashes for the entire study period could not be easily reached since the number of crash occurrences was small.

5.4 Recommendations

1. Estimated calibration factors and developed calibration functions as provided in Table 5.1, can be used to predict crashes more accurately for Kansas freeway and ramp facilities. Since calibration functions showed a good reliability for freeway segments, speed-change lanes, and crossroad ramp terminals, it is acknowledged that these calibration functions are used in crash predictions.
2. Estimated disaggregate calibration factors and developed disaggregate calibration functions by each area type and cross section type, as shown in Table 4.1 through Table 4.4, can be used to predict crashes more accurately for each freeway type, speed-change lane type, ramp type, and crossroad ramp terminal type.
3. It is recommended that Kansas-specific crash type distributions for freeway segments, speed-change lanes, and crossroad ramp terminals be used to identify types of crashes that could occur at these facilities in such a way that necessary treatments can be implemented.
4. It is recommended that Kansas-specific SPFs be developed for multiple vehicle crash models in exit ramp segments, as both fatal and injury and property damage only models reported too-high calibration factors and the number of reported ramp crashes during the study period was insufficient to develop reliable calibration functions.

5. This study recommends considering incorporating crash locations, such as ramps, speed-change lanes, C-D roads, and crossroad ramp terminals, when revising existing motor vehicle crash reports for a certain jurisdiction, so that these crashes are accurately reported in future years.
6. This study also recommends considering the inclusion of geometric data types needed for freeway and ramp facility calibrations to state-maintained electronic databases.
7. It is important to ensure that police officers are aware of these freeway and ramp crash locations and terminology when reporting interchange-related crashes so that these crashes are accurately reported for future years.

5.5 Limitations of the Study

Since the value of calibration factors rely completely on the number of reported crashes, a certain possibility exists of resulting inaccurate values for these calibration factors. However, not only this study, but all crash-related studies undergo the issue of uncertainty in accuracy and completeness in reported crashes. For example, police officers may not have been notified if either road users agree to sign private informal agreements for insurance purposes, no third parties were involved during the crash (i.e., single vehicle crashes), or no obvious injuries occurred as a result of the crash (Amoros, Martin, & Laumon, 2006). Moreover, miscoding, misreporting, and incompleteness are three main issues in electronic crash databases and therefore, in reality, reported crash data are not perfect either.

In addition, a large majority of geometric data required for ramp segment and crossroad ramp terminal calibration were collected using Google Earth. The research team did its best to collect accurate geometric data; however, the possibility exists of getting imprecise numbers for geometries. For example, in some cases, measuring the length and offset of the median/outside barrier using the ruler in Google Earth was difficult due to unclear views caused by oversized trucks. Lastly, a manual review of 13,750 crash reports was performed in this study and accordingly, the possibility of human error exists in this extensive crash report review process.

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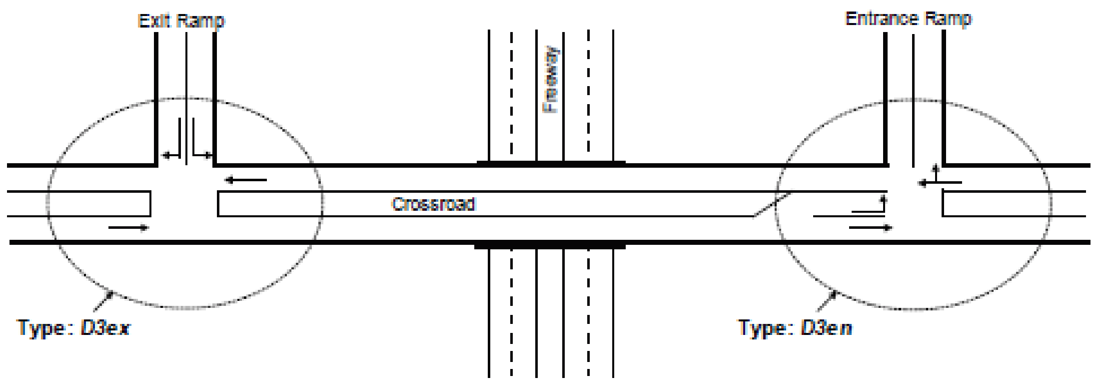
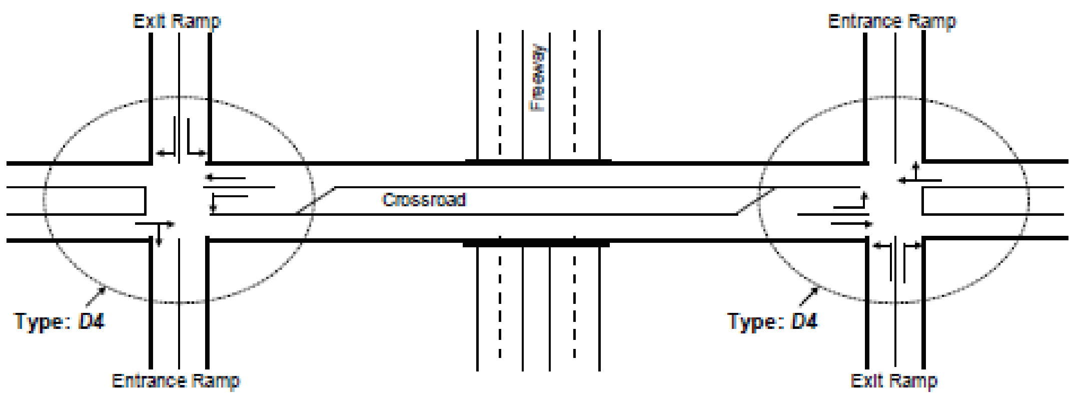
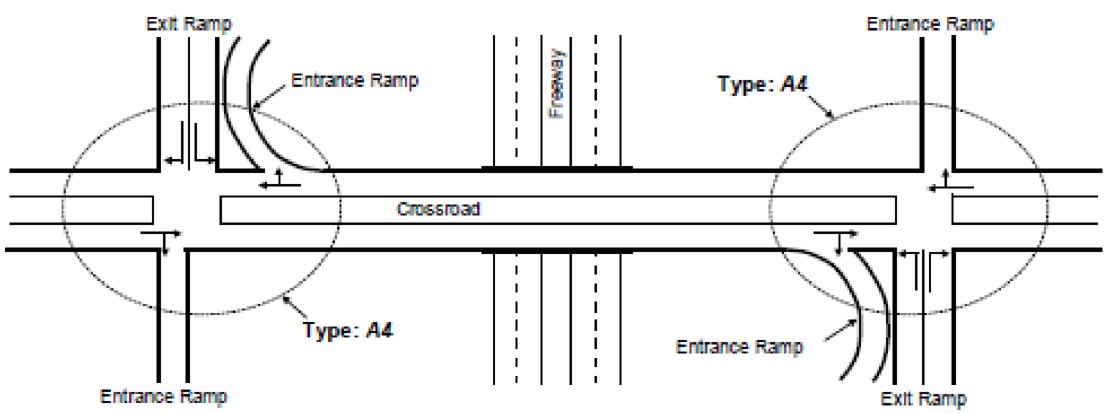
Appendix A: Crossroad Ramp Terminal Configurations Covered in the HSM



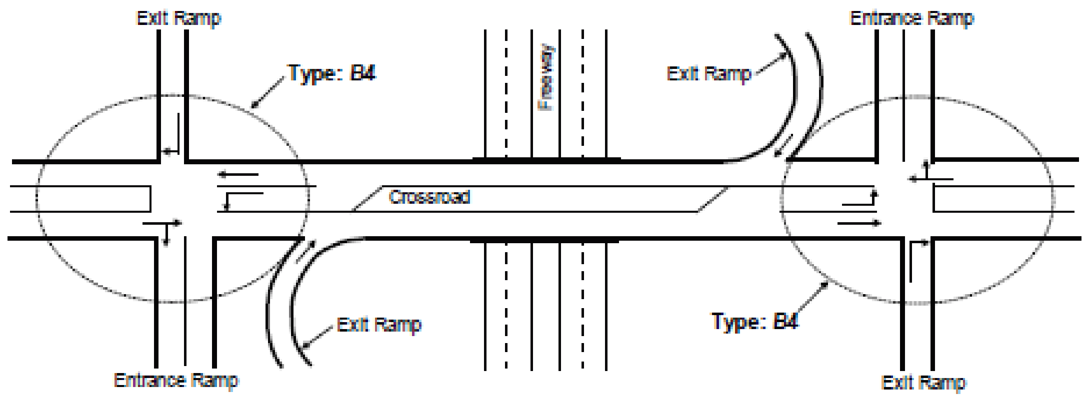
a. Three-Leg Ramp Terminal with Diagonal Exit or Entrance Ramp (*D3ex* and *D3en*)



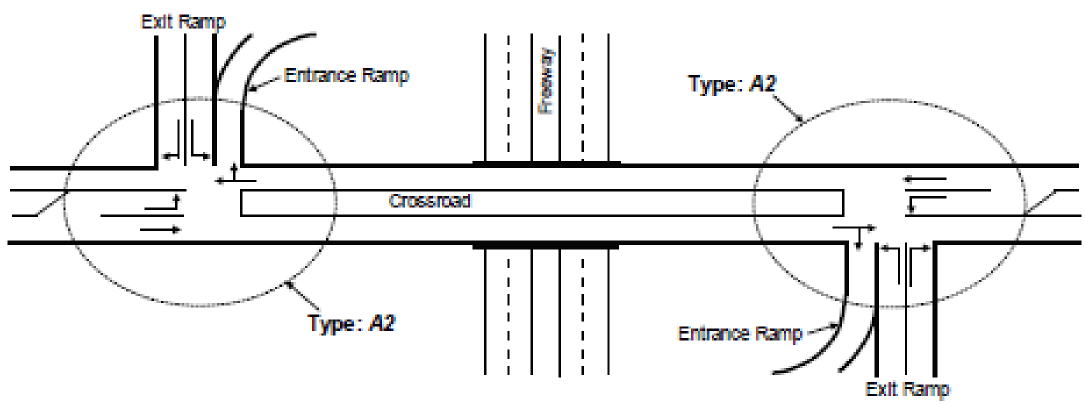
b. Four-Leg Ramp Terminal with Diagonal Ramps (*D4*)



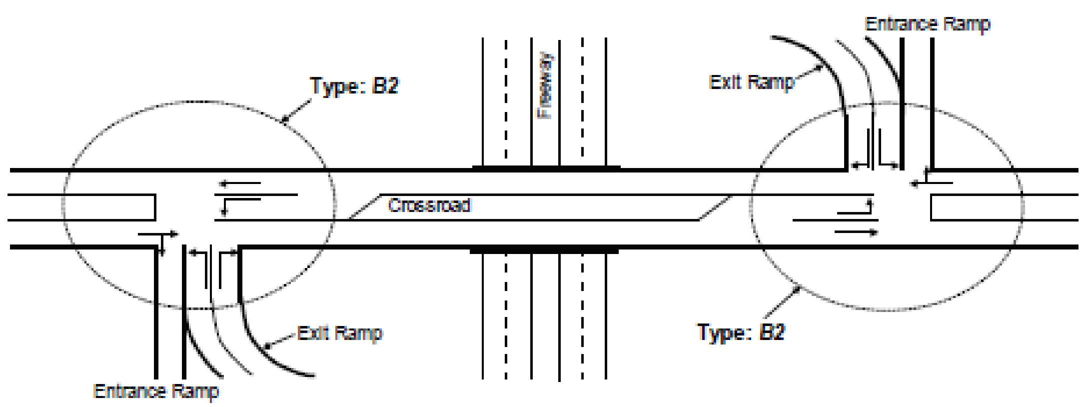
c. Four-Leg Ramp Terminal at Four-Quadrant Partial Cloverleaf A (*A4*)



d. Four-Leg Ramp Terminal at Four-Quadrant Partial Cloverleaf B (*B4*)

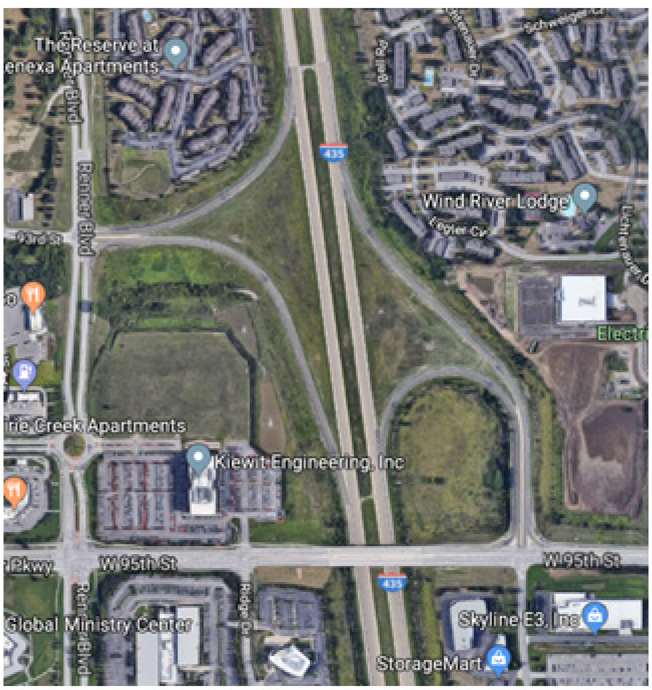


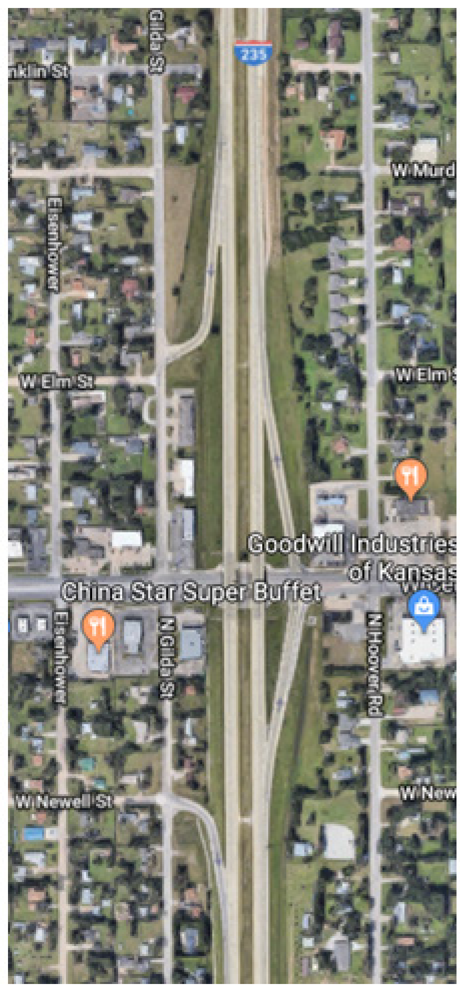
e. Three-Leg Ramp Terminal at Two-Quadrant Partial Cloverleaf A (*A2*)



f. Three-Leg Ramp Terminal at Two-Quadrant Partial Cloverleaf B (*B2*)

Appendix B: Unique Crossroad Ramp Terminal Configurations Present in Kansas





Appendix C: Sample Calculation of Freeway Segment Calibration Factors

Site/Freeway Facility: A rural 4-lane freeway segment with a horizontal curve

The facts:

The study period is 2013 to 2015. The conditions present during this period are provided below.

- 0.42 in length
- AADT (year) = 11,800 vpd (2013), 12,600 vpd (2014), 13,000 vpd (2015)
- One horizontal curve
    - 17,198 ft radius in one roadbed and 17,282 in the second roadbed
    - 0.42 mi in length – entirely in the segment
    - Curve exits in both road beds
- 12 ft lane width
- 10 ft outside shoulder width
- 6 ft inside shoulder width
- 40 ft median width
- Rumble strips present on inside and outside shoulders entirely in the segment
- No median barrier or roadside barrier
- No type B viewing sections
- 30 ft clear zone width

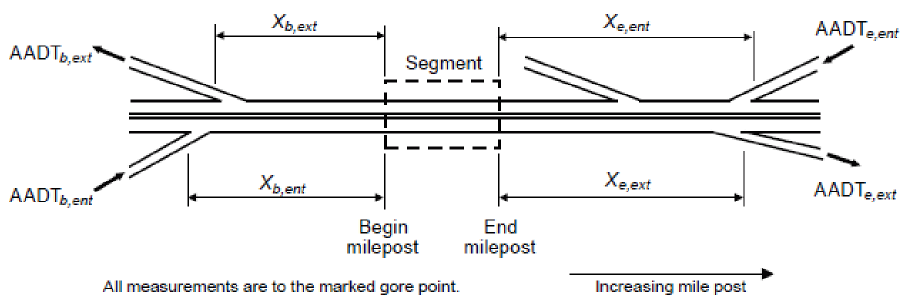


Table C.1: Data to Describe Four Ramps in the Vicinity of the Segment

Variable Script	b,ent	e,ext	e,ent	b,ext
Distance from segment, Xa,b (mi)	1.35	0.08	0.07	1.35
Ramp Volume, AADTa,b (2015) (vpd)	50	465	470	39
Ramp Volume, AADTa,b (2014) (vpd)	21	500	500	10
Ramp Volume, AADTa,b (2013) (vpd)	21	500	500	10

Note: Data in Table C.1 were collected using ArcGIS measure tools and GIS ramp shapefiles as mentioned in Section 3.6.

CMF 1 – Horizontal Curve

The base condition is an uncurved (tangent segment).

$$CMF_{1,fs,ac,y,z} = 1 + a \times \left[\sum_{i=1}^{m} \left(\frac{5730}{R_i^*} \right)^2 \times P_{c,i} \right]$$

Where,

$CMF_{,1,fs,ac,y,z}$	=	Crash modification factor for horizontal curvature in a freeway segment with any cross section ac, crash type y, and severity z
R_i^*	=	Equivalent radius of curve i (ft) = [0.5/$R_{a,i}$ ^2 + 0.5/$R_{b,i}$ ^2], if both roadbeds are curved, $R_{a,i}$ if only one roadbed is curved
$R_{a,i}$	=	Radius of curve i in one roadbed (ft)
$R_{b,i}$	=	Radius of curve i in second roadbed (ft) *if both roadbeds are curved
$P_{c,i}$	=	Proportion of effective segment length with curve i
m	=	Number of horizontal curves in the segment

The segment is 0.42 mi long, the curve is present throughout the entire segment length in both roadbeds, and its entire length is in the segment. Hence, $P_{c,i}$ = 1.00 and R_i^* = 17, 240 ft. From the HSM Table 18-14, a = 0.0172 for multiple vehicle fatal and injury crashes (AASHTO, 2014, pp. 18-36). The $CMF_{1,fs,4,mv,fi}$ is calculated as follows:

$$CMF_{1,fs,4,mv,fi} = 1 + 0.0172 \times \left[\sum_{i=1}^{m} \left(\frac{5730}{17{,}240} \right)^2 \times 1 \right] = 1.0019$$

The calculations using the other coefficients from the HSM Table 18-14 provides the following results:

$$CMF_{1,fs,4,mv,pdo} = 1.0038$$

$$CMF_{1,fs,4,sv,fi} = 1.0079$$

$$CMF_{1,fs,4,sv,pdo} = 1.0069$$

CMF 2 – Lane Width

The base condition is a 12 ft lane width.

The segment has 12 ft lanes, which is the base condition for the lane width CMF. So, $CMF_{2,fs,4,y,fi}$ and $CMF_{2,fs,4,y,pdo}$ are equal to 1.0000. If the lane width is different from 12 ft, use the HSM Equation 18-25 to calculate the relevant CMF (AASHTO, 2014, pp. 18-36).

CMF 3 – Inside Shoulder Width

The base condition is a 6 ft inside shoulder width.

The segment has 6 ft inside shoulders, which is the base condition for the inside shoulder CMF. So, $CMF_{3,fs,4,y,fi}$ and $CMF_{3,fs,4,y,pdo}$ are equal to 1.0000. If the inside shoulder width is different from 6 ft, use the HSM Equation 18-26 to calculate the relevant CMF (AASHTO, 2014, pp. 18-37).

CMF 4 – Median Width

The base condition is a 60 ft median width, a 6 ft inside shoulder width, and no barrier present in the median.

$$CMF_{4,fs,4,y,z} = (1 - P_{ib}) \times exp(a \times [W_m - 2 \times W_{is} - 48]) + P_{ib} \times exp(a \times [2 \times W_{icb} - 48])$$

Where,

$CMF_{,4,fs,ac,y,z}$	=	Crash modification factor for median width in a freeway segment with any cross section *ac*, crash type *y*, and severity
P_{ib}	=	Proportion of effective segment length with a barrier present in the median (i.e., inside)
W_m	=	Median width (ft) -measured from near edges of traveled way in both directions
W_{icb}	=	Distance from edge of inside shoulder barrier to barrier face (ft)

The segment does not have a median barrier, therefore, P_{ib} = 0.0 and the calculation of W_{icb} does not apply. From the HSM Table 18-17, a = -0.00305 for multiple vehicle fatal and injury crashes (AASHTO, 2014, pp. 18-38). The $CMF_{4,fs,4,mv,fi}$ is calculated as follows:

$$CMF_{4,fs,4,mv,fi} = (1 - 0) \times exp(-0.00302 \times [40 - 2 \times 6 - 48]) + 0.0 \times exp(-0.00302 \times [2 \times W_{icb} - 48]) = 1.062$$

The calculations using the other coefficients from the HSM Table 18-17 provides the following results:

$CMF_{4,fs,4,mv,pdo} = 0.980$

$CMF_{4,fs,4,sv,fi} = 1.060$

$CMF_{4,fs,4,sv,pdo} = 1.060$

CMF 5 – Median Barrier

The base condition is no median barrier present.

The segment does not have a median barrier, which is the base condition for the median barrier CMF. So, $CMF_{5,fs,4,y,fi}$ and $CMF_{5,fs,4,y,pdo}$ are equal to 1.0000. If a median barrier is present within the segment, use the HSM Equation 18-28 to calculate the relevant CMF (AASHTO, 2014, pp. 18-38).

CMF 6 – High Volume

The base condition is no hours having a volume exceeds 1,000 veh/hr/ln.

As the study did not have any data on the number of hours where volume exceeds 1,000 veh/hr/ln, the HSM equation 18-29 was used to estimate the CMF for High Volume (AASHTO, 2014, pp.18-39, pp. 18-21). From the HSM Table 18-19, a = 0.35 for multiple vehicle fatal and injury crashes (AASHTO, 2014, pp. 18-40).

$$CMF_{6,fs,4,y,z} = exp(a \times P_{hv})$$

$$P_{hv} = 1 - exp\left(1.45 - 0.000124 \times {AADT}/{n}\right)$$

Where,

$CMF_{6,fs,ac,y,z}$	=	crash modification factor for high volume in a freeway segment with any cross section ac, crash type y, and severity z
P_{hv}	=	Proportion of AADT during hours where volume exceeds 1,000 veh/hr/ln, *if the value computed is less than 0.0, then set it to 0.0.
n	=	Number of through lanes (i.e. 4, 6, 8, and 10)

The $CMF_{6,fs,4,mv,fi}$ is calculated as follows:

$$P_{hv,2013} = 1 - exp\left(1.45 - 0.000124 \times {}^{11{,}800}/_{4}\right) = -1.9570$$

$$P_{hv,2014} = 1 - exp\left(1.45 - 0.000124 \times {}^{12{,}600}/_{4}\right) = -1.8864$$

$$P_{hv,2015} = 1 - exp\left(1.45 - 0.000124 \times {}^{13{,}000}/_{4}\right) = -1.8490$$

The calculated P_{hv} values were less than 0.0 for all three years. Therefore, the value P_{hv} was set as 0.0.

$$CMF_{6,fs,4,mv,fi\ (2013-2015)} = exp(0.35 \times 0) = 1.000$$

So, $CMF_{6,fs,4,y,fi\ (2013\text{-}2015)}$ and $CMF_{6,fs,4,y,pdo\ (2013\text{-}2015)}$ are equal to 1.0000.

CMF 7 – Lane Change

The base condition is no significant lane changing due to ramp entry or exit. More specifically no ramp entrance or exit within 0.5 mi of the segment.

$$CMF_{7,fs,4,y,z} = \left(0.5 \times f_{wev,inc} \times f_{lc,inc}\right) + \left(0.5 \times f_{wev,dec} \times f_{lc,dec}\right)$$

Where,

$$f_{wev,inc} = \left(1 - P_{wevB,inc}\right) \times 1.0 + P_{wevB,inc} \times exp\left(\frac{a}{L_{wev,inc}}\right)$$

$$f_{wev,dec} = \left(1 - P_{wevB,dec}\right) \times 1.0 + P_{wevB,dec} \times exp\left(\frac{a}{L_{wev,dec}}\right)$$

$$f_{lc,inc} = \left(1.0 + \frac{exp\left(-b \times X_{b,ent} + d \times ln\left[c \times AADT_{b,ent}\right]\right)}{b \times L_{fs}} \times \left[1.0 - exp\left(-b \times L_{fs}\right)\right]\right) + \left(1.0 + \frac{exp\left(-b \times X_{e,ext} + d \times ln\left[c \times AADT_{e,ext}\right]\right)}{b \times L_{fs}} \times \left[1.0 - exp\left(-b \times L_{fs}\right)\right]\right)$$

$$f_{lc,dec} = \left(1.0 + \frac{exp\left(-b \times X_{e,ent} + d \times ln\left[c \times AADT_{e,ent}\right]\right)}{b \times L_{fs}} \times \left[1.0 - exp\left(-b \times L_{fs}\right)\right]\right) + \left(1.0 + \frac{exp\left(-b \times X_{b,ext} + d \times ln\left[c \times AADT_{b,ext}\right]\right)}{b \times L_{fs}} \times \left[1.0 - exp\left(-b \times L_{fs}\right)\right]\right)$$

Where,

$CMF_{,7,fs,mv,y,z}$	=	Crash modification factor for lane changes in a freeway segment with any cross section ac, multiple- vehicle crashes mv, and severity z
$f_{lc,inc}$	=	Lane change adjustment factor for travel in increasing milepost direction
$f_{lc,dec}$	=	Lane change adjustment factor for travel in decreasing milepost direction
$f_{wev,inc}$	=	Weaving section adjustment factor for travel in increasing milepost direction
$f_{wev,dec}$	=	Weaving section adjustment factor for travel in decreasing milepost direction
$P_{wevB,inc}$	=	Proportion of segment length within a Type B weaving section for travel in increasing milepost direction
$P_{wevB,dec}$	=	Proportion of segment length within a Type B weaving section for travel in decreasing milepost direction
$L_{wev,inc}$	=	Weaving section length for travel in increasing milepost direction (may extend beyond segment boundaries) (mi)
$L_{wev,dec}$	=	Weaving section length for travel in increasing milepost direction (may extend beyond segment boundaries) (mi)
$X_{b,ent}$	=	Distance from segment begin milepost to nearest upstream entrance ramp gore point, for travel in increasing milepost direction (mi)
$X_{b,ext}$	=	Distance from segment begin milepost to nearest downstream exit ramp gore point, for travel in decreasing milepost direction (mi)
$X_{e,ent}$	=	Distance from segment begin milepost to nearest upstream entrance ramp gore point, for travel in decreasing milepost direction (mi)
$X_{e,ext}$	=	Distance from segment begin milepost to nearest downstream exit ramp gore point, for travel in increasing milepost direction (mi)
$AADT_{b,ent}$	=	AADT volume of entrance ramp located at distance $X_{b,ent}$ (vpd)
$AADT_{b,ext}$	=	AADT volume of exit ramp located at distance $X_{b,ext}$ (vpd)
$AADT_{e,ent}$	=	AADT volume of entrance ramp located at distance $X_{e,ent}$ (vpd)
$AADT_{e,ext}$	=	AADT volume of exit ramp located at distance $X_{e,ext}$ (vpd)

Table C.1 provides the required data elements needed to estimate the CMF for lane change. This was the most complex CMF in the freeway segment calibration due to extensive details. This segment does not have Type B weaving sections, therefore the weaving adjustment factors $f_{wev,inc}$ and $f_{wev,dec}$ are equal to 1.00. Following shows the $f_{lc,inc}$ and $f_{lc,dec}$ calculation for the year 2013. From the HSM Table 18-20, a = 0.175, b = 12.56, c = 0.001, and d = -0.272 for multiple vehicle fatal and injury crashes (AASHTO, 2014, pp. 18-42).

$$f_{lc,inc} = \left(1.0 + \frac{exp(-12.56 \times 1.35 - 0.272 \times ln[0.001 \times 21])}{-12.56 \times 0.42} \times [1.0 - exp(-12.56 \times 0.42)]\right) + \left(1.0 + \frac{exp(-12.56 \times 0.08 - 0.272 \times ln[0.001 \times 500])}{-12.56 \times 0.42} \times [1.0 - exp(-12.56 \times 0.42)]\right) = 1.0000$$

$$f_{lc,dec} = \left(1.0 + \frac{exp(12.56 \times 0.07 - 0.272 \times ln[0.001 \times 500])}{-12.56 \times 0.42} \times [1.0 - exp(-12.56 \times 0.42)]\right) + \left(1.0 + \frac{exp(-12.56 \times 1.35 - 0.272 \times ln[0.001 \times 10])}{-12.56 \times 0.42} \times [1.0 - exp(-12.56 \times 0.42)]\right) = 1.0166$$

$$CMF_{7,fs,4,mv,fi\ (2013)} = (0.5 \times 1.00 \times 1.0000) + (0.5 \times 1.00 \times 1.0166) = 1.0083$$

Similarly, the calculations using the other coefficients from the HSM Table 18-19 provides the following results:

$CMF_{7,fs,4,mv,pdo\ (2013)} = 1.0073$

$CMF_{7,fs,4,mv,fi\ (2014)} = 1.0083$

$CMF_{7,fs,4,mv,pdo\ (2014)} = 1.0073$

$CMF_{7,fs,4,mv,fi\ (2015)} = 1.0085$

$CMF_{7,fs,4,mv,pdo\ (2015)} = 1.0075$

CMF 8 – Outside Shoulder Width

The base condition is a 10 ft outside shoulder width.

The segment has 10 ft outside shoulders, which is the base condition for the outside shoulder width CMF. Hence, $CMF_{8,fs,4,sv,fi}$ and $CMF_{8,fs,4,sv,pdo}$ are equal to 1.0000. If the outside shoulder width is different from 10 ft, use the HSM Equation 18-35 to calculate the relevant CMF (AASHTO, 2014, pp. 18-42).

CMF 9 – Shoulder Rumble Strips

The base condition is no shoulder rumble strips present.

The proportion P_{ir} or P_{or} represents the proportion of the effective segment length with rumble strips present on the inside shoulders or outside shoulders. It is computed by summing the length of roadway with rumble strips on the inside shoulder or outside shoulder (excluding the length of any rumble strips adjacent to speed-change lanes) in both travel directions and dividing by twice the effective freeway segment length.

The segment has shoulder rumble strips on both the inside and outside shoulders throughout the entire segment length in both directions of travel. Therefore, P_{ir} and P_{or} equals to 1.00.

$$CMF_{9,fs,4,sv,fi} = \left(1.0 - \sum_{i=1}^{m} P_{c,i} \times f_{c,i}\right) \times f_{tan} + \left(\sum_{i=1}^{m} P_{c,i} \times f_{c,i}\right) \times 1.0$$

$$f_{tan} = 0.5 \times ([1.0 - P_{ir}] \times 1.0 + P_{ir} \times 0.811) + 0.5 \times ([1.0 - P_{or}] \times 1.0 + P_{or} \times 0.811)$$

Where,

$CMF_{9,fs,ac,sv,fi}$	=	Crash modification factor for shoulder rumble strips in a freeway segment with any cross section ac and fatal-and-injury (fi) single-vehicle (sv) crashes
f_{tan}	=	Factor for rumble strip presence on tangent portions of the segment;
P_{ir}	=	Proportion of effective segment length with rumble strips present on the inside shoulders
P_{or}	=	Proportion of effective segment length with rumble strips present on the outside shoulders

$$f_{tan} = 0.5 \times ([1.0 - 1.0] \times 1.0 + 1 \times 0.811) + 0.5 \times ([1.0 - 1.0] \times 1.0 + P_{or} \times 0.811) = 0.811$$

$$CMF_{9,fs,4,sv,fi} = 0.8110$$

CMF 10 – Outside Clearance

The base condition is a 30 ft clear zone, a 10 ft outside shoulder width and no barrier present at the clear zone.

The segment has a clear 30 ft clear zone, a 10 ft outside shoulder width and no barrier present at the clear zone. So, $CMF_{10,fs,4,sv,fi}$ and $CMF_{10,fs,4,sv,pdo}$ are equal to 1.0000. If any of the variables change from the set base conditions, use the HSM Equation 18-38 to calculate the relevant CMF (AASHTO, 2014, pp. 18-44).

CMF 11 – Outside Barrier

The base condition is no barrier present in the clear zone.

The segment does not have outside barriers. So, $CMF_{11,fs,4,sv,fi}$ and $CMF_{11,fs,4,sv,pdo}$ are equal to 1.0000. If an outside barrier present in the segment, use the HSM Equation 18-39 to calculate the relevant CMF (AASHTO, 2014, pp. 18-44).

In summary, it can be noted that calculation of the freeway segment CMFs were extremely challenging. Furthermore, the HSM also provides supplemental calculations to obtain CMF 4, CMF 5, CMF 10, and CMF 11 in a separate section as the equations are very lengthy (AASHTO, 2014, pp. 18-50).

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