Evaluating Local and Tribal Rural Road Design with the Interactive Highway Safety Design Model (IHSDM)

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Disclaimer

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ABSTRACT

Establishing performance-based safety goals and objectives becomes more attainable with the Highway Safety Manual (HSM). However, the safety performance functions (SPFs) in the HSM may not be accurate as they are not calibrated to local conditions. In addition, each SPF and crash modification factor (CMF) assumes a set of base site conditions which may not be realistic for local roadways. Although calibration procedures are available in HSM Part C Appendix A, they should be refined or modified to accommodate local data availability and roadway, traffic, and crash characteristics. It is also necessary to determine a set of base conditions applicable to local highways. This document presents the application of the HSM for rural local two-lane two-way highway segments in South Dakota (SD). The calibration was based on three-year (2009-2011) crash data from 657 roadway segments constituting more than 750 miles of roadways. The calibration process includes establishing new base conditions, developing SPFs, converting CMFs to base conditions as well as substituting default values with state-specific values. Five models have been developed and compared based on statistical goodness-of-fit and calibration factors. The same procedures were also conducted for the tribal two-lane two-way highway segments in SD based on three-year (2009-2011) crash data from 56 roadway segments constituting 199.5 miles of roadway.

Results show that the jurisdiction-specific crash type distribution for CMFs can be drastically different from what is presented in the HSM. For rural local two-lane two-way roadways, the HSM method without modification underestimates SD crashes by 35 percent. The method based on SPFs developed from a full model has the best performance. For tribal two-lane two-way roadways, the HSM method without modification overestimates SD crashes by 122 percent. The method using the exponential from of annual average daily traffic (AADT) performs the best. This documentation provides important guidance and empirical results regarding how to calibrate HSM models.

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LIST OF ACRONYMS AND ABBREVIATIONS

AADT	Average Annual Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
BIA	Bureau of Indian Affairs
C/ Cr	Calibration Factor
CMF	Crash Modification Factor
CPM	Crash Prediction Module
FARS	Fatality Analysis Reporting System
FHWA	Federal Highway Administration
GIS	Geographic Information System
GPS	Global Positioning System
HSM	Highway Safety Manual
MAD	Mean absolute deviation
ND	North Dakota
NDDOT	North Dakota Department of Transportation
PDO	Property Damage Only
PRM	Policy Review Module
RIS	Roadway Inventory System
SD	South Dakota
SDDOT	South Dakota Department of Transportation
SPF	Safety Performance Function
VPI	Vertical Point Intersection

1. INTRODUCTION

Recent research on rural roads has highlighted safety challenges and opportunities.[1,2,3] To bridge the knowledge gap between practice and research, safety improvement decision support tools have been developed such as SafetyAnalyst and Interactive Highway Safety Design Module (IHSDM). It is anticipated that the transportation agencies aided by these tools will make decisions more effectively and efficiently. Research proposed in this study serves the need to understand the requirements, limitations, and performance of these tools. Specifically, IHSDM was evaluated and calibrated with available safety data to improve the safety decisions for local and tribal roads in South Dakota (SD) and North Dakota (ND).

Local road crashes are a substantial safety issue in the Dakotas. According to the Fatality Analysis Reporting System (FARS), Native Americans accounted for 26% of all traffic fatalities from 2001 to 2005 in South Dakota.[4] The motor vehicle fatality rate of Native Americans is more than three times higher than other South Dakotans. The safety statistics are similar in North Dakota. It is imperative to create a safer driving environment for the tribal lands and local roads in North Dakota and South Dakota by implementing improved safety standards in planning, designing and building roads.

Many tribal communities are in rural areas and are connected by two-lane rural highways. The design of these roads may be substandard, i.e., some of them were not necessarily designed by engineers. The road surface condition can be rapidly damaged under the effects of weather and heavy traffic loads. The safety of these roads may be further reduced by insufficient pavement marking and signage, especially with regard to narrow road width in the absence of shoulders. The safety need of these rural low-volume roads is pressing; 40% of fatal crashes in the United States occur on local roads. Rural roads also have much higher crash rates than urban roads, which led to the establishment of the High Risk Rural Roads (HRRR) program in SAFETEA-LU.[5] Among all rural highways, the safety research in unpaved roads is most limited. A few pertinent publications regarding safety of unpaved roads can be found from Iowa, Wyoming, and Kansas. Kansas researchers found that environment and road factors had more influences on crash occurrence on gravel roads than on paved roads.[6,7] In a recently published Iowa report, a detailed comparison between crashes on paved low-volume roads (less than 400 vehicles per day) and unpaved roads was conducted and particular interest was directed to unpaved rural roads with traffic volumes greater than 100 vehicles per day.[8]

Local and tribal agencies have committed to improving local traffic safety by reducing the number and severity of crashes. However, these agencies have long faced the dilemma of addressing safety concerns on their roadways while balancing available funds. Moreover, defining safety performance expectations is a challenge for local and tribal transportation agencies. The Highway Safety Manual (HSM) of the American Association of State Highway and Transportation Officials (AASHTO) provides guidance for safety analysis using scientific and statistically sound methods.[9] Given the expense of engineering studies and limited funding, safety reviews based on expected safety performance are a useful way to identify hot spots in a highway network as well as site-specific safety problems. Predictive crash models, as formulated in Equation1, pinpoint sites with great promise for crash reduction on the basis of decades of safety research and statistical analysis.

$$N_{predicted} = N_{spf} \times C \times (CMF_1 \times \cdots \times CMF_n) \qquad (1)$$

Where $N_{predicted}$ is the predicted average crash frequency for a site, N_{spf} is the predicted average crash frequency for base conditions for a site, also called safety performance function (SPF), and C is the calibration factor (C_r is for a roadway segment and C_i is for an intersection). A series of crash modification factors (CMFs) account for changes in the number of crashes due to specific site characteristics or safety treatments. Locations where the actual crash count is higher than the predicted crash count need to be further investigated for safety improvements.

Because road, environment, driver and other conditions in South Dakota may not be identical to those of the states used to develop HSM, agencies should not use HSM models without calibration. Un-calibrated models compromise safety estimates, produce unrealistic results, and undermine accountability. Even agencies that use their own data to develop SPFs should consider calibrating the models every two to three years because safety conditions change over time. HSM models must be calibrated for the results to be comparable to the estimates obtained from an agency's records. Although calibration procedures are available in HSM Appendix A, they need to be refined or modified to accommodate data availability and roadway, traffic, and crash characteristics.

Though safety can be improved by repairing and fixing hazardous road locations, these remedial activities are only simple patches to the roadway network. It has been widely accepted in recent decades that a significant portion of crashes could have been avoided by a systematic, consistent, and proactive approach to road planning, design and construction. The Interactive Highway Safety Design Model (IHSDM) is a proactive tool in addressing safety issues during the road design stages – before the project is constructed.[10] IHSDM crash prediction models are a faithful implementation of Highway Safety Manual Part C methodology. More recently, the latest update of 2011 AASHTO "Green Book" policies for rural two-lane highways were added to the new release of IHSDM-HSM Predictive Method 2011. According to the feedback about using IHSDM in several pilot studies conducted in PA, KY, and UT, extensive data collection is needed and model adjustment and calibration is necessary.[11, 12, 13, 14] The challenge of collecting essential data components and calibrating model estimates highlights to the need for this research project which will provide a reliable and realistic estimation for highway safety in project planning and design.

The objectives of this study are to address the potential challenges and obstacles to implementing safety design modules; modify, validate and calibrate IHSDM for local conditions; and apply the refined modules to compare design alternatives.

2. LITERATURE REVIEW

Decades of research and practices have demonstrated that highway safety can be substantially improved by a scientific, systematic, consistent, and proactive approach to highway planning and design. The Interactive Highway Safety Design Model (IHSDM) is a proactive tool to support project-level geometric design decisions by providing quantitative information on the expected safety and operational performance before project construction.[10] The Crash Prediction Module in IHSDM allows planners, designers, and reviewers to comprehensively assess the expected safety performance of design plans via Federal Highway Administration (FHWA) endorsed methodologies in the Highway Safety Manual.[9] The Policy Review Module seamlessly incorporates safety consideration into roadway design by checking the compliance of design plans to AASHTO standards. To exercise the full potential of IHSDM for a realistic and reliable estimate of crashes for local roads with certain geometric and traffic characteristics, the tool needs to be adjusted and calibrated. If properly designed, the calibration is able to account for the disparities between the national crash predictive models and local crash data.

Calibrating IHSDM models is essentially the same as calibrating the crash predictive models in HSM because IHSDM models are faithful implementations of HSM. The calibration process accounts for the different safety effects due to driver population, environmental variables, and other unobserved or unmeasured factors. In HSM, predicting the number of crashes for an entity given a set of values for input variables follows a three-step process. Starting with a Safety Performance Function (SPF), Crash Modification Factors or Functions (CMFs) and calibration factors (Cr) are subsequently followed. SPF predicts the crash frequency as a function of AADT for roadway segment with basic geometric and traffic conditions. The rural two-lane, two-way road SPF used in this study was formulated assuming base conditions for the highway facility which include: 12-foot lane width, 6-foot shoulder width and paved shoulder, 3-point of roadside hazard rating, five driveways per mile, level grade with no horizontal curvature, no vertical curvature, no centerline rumble strips, no passing lanes, no two-way left-turn lanes, no lighting, and no automated speed enforcement. It is expected that the crash density of roadway segments satisfying the base condition varies as the power function of AADT. Note that not many sites on rural local roads have paved shoulders, let alone a 6-foot paved shoulder; and not many rural local roads are without horizontal curves either. For sites possessing different characteristics than base conditions, CMFs can be multiplied to the crash frequency at base conditions with each CMF representing one type of change. Once all available CMFs have been considered, a calibration factor Cr serves as the ultimate adjustment for all the other known or unknown, measurable or immeasurable differences, such as climate, driver populations, animal populations, crash reporting thresholds, and crash reporting system procedures. Cr is the ratio of the expected number of crashes and observed number of crashes. Each of the three steps yields the opportunity for calibration if more accurate results are desired.

Since the SPF carries the majority of weight in predicting crashes, calibrating the SPF may be more critical and effective than other modifications. Banihashemi[15] used the roadway segments satisfying the base conditions to develop two respective base models as $L \times AADT$ and $L \times AADT^{p}$, where L is the segment length and $AADT$ is the annual average daily traffic, and p is power coefficient. In general, the base conditions should be the most representative segment among all types, which guarantee a sizable sample for developing statistically robust models. However, the most representative roadway type may vary from state to state, region to region. Because of this, it may be questionable for any state to justify the same base conditions. For a small sample size satisfying base conditions, SPF calibration may not be rigorous enough or represent the larger population. The resulting estimation deviance will be further amplified after applying CMFs.

An alternative method employed by Martinelli and et al.[16] was to develop the predictive crash models by using the full model proposed in the HSM prototype chapter:[17, 18] a model with variables including segment length, AADT, lane width, shoulder width, roadside hazard rating, driveway density, horizontal curvature, grade rate for crest vertical curves, and percent grade for straight grades was developed, and then substituted values of variables corresponding to base conditions except for AADT and segment length to obtain the SPF. This method seems to address the sampling issue of sites meeting the base conditions but is essentially the development of a full-scale crash prediction model, which makes the subsequent steps of CMFs and Cr redundant or even unnecessary. Developing a full-scale model is certainly more complicated than developing a model with just AADT and length.[19] Model specification, variable correlation, and interaction need to be carefully considered when more variables are involved. In this study, following HSM recommended calibration process instead of developing a full-scale model is considered.[9]

After the SPF is calibrated, CMFs are multiplied to SPF. CMFs could be in the format of a factor or a function. For instance, a CMF for centerline rumble strips is a factor, whereas a CMF for a horizontal curve is a function of curve length and degree of curve. Sun et al.[20] created a database that has the most important variables such as segment length, ADT, lane width, and shoulder type and width, all of which are available, while setting other variables as the same as base conditions. When combined with the empirical Bayes method and calibration parameter as a function of ADT, it is found that the result is satisfactory, for the differences between the observed and predicted crash frequencies are well within the 5% range. Sensitivity analysis was performed by collecting additional data to test the effects of driveway density and horizontal curves. It is concluded that omitting one or more insignificant variables in model calibration won't compromise model's accuracy but helps to alleviate the burden of collecting additional data from a practical standpoint.

Both the SPF and CMFs account for the safety effects of measured variables. The unmeasured factors can be estimated via an overall calibration factor or function. A calibration factor, Cr, is the ratio of the expected crash frequency (calculated by the SPF multiplied by all the available CMFs) to the observed crash frequency. Cr can be directly estimated through IHSDM once the data is imported. HSM also recommends using Cr to adjust for regional differences.[16, 21] However, when suspecting that the unmeasured errors or factors may be correlated with observable variables such as AADT, the calibration function can be more effective in describing the trend or pattern than a single ratio. Sun et al.[20] treated the calibration parameter as a function of AADT and set different calibration values for different ranges of AADT. Compared to the large number of papers adopting the single value as the calibration factor,[16, 21, 22] there is little research on the calibration function, or proof that a well-defined calibration function can greatly improve the predictive power of the crash prediction models, for the trend of the ratio of the estimation to observation may change considerably; which is difficult to be represented by a single value. In this study, we investigated AADT, segment length, and other parameters as the function of calibration factor. Given the low AADT on local and tribal roads, the segment length or other length related factors may prominently affect safety more than traffic volume.

3. DATA COLLECTION

The literature review provides a comprehensive review of state-of-the-art HSM calibration methodologies, data needs, and requirements. To use IHSDM, road data and crash data related to the roads meeting IHSDM data requirement are both needed. The data was obtained from different sources and prepared to run IHSDM with some assumptions. Of the six modules in IHSDM, two were of interest: the Policy Review Module (PRM) and the Crash Prediction Module (CPM). PRM checks a design relative to the range of values for critical dimensions recommended in AASHTO design policy. CPM provides estimates of expected crash frequency and severity.

3.1 Roadway Data Collection

3.1.1 IHSDM Data Requirement

The required data in the crash prediction module include:

- Horizontal alignment
- Vertical alignment
- Lane characteristics
- Cross slope
- Annual average daily traffic
- Design speed

The optional data in the module is not necessary, but if it is available, along with required data, the roadway can be represented more precisely.

The required and optional data in the crash prediction module is listed in Table 3.1 and Table 3.2.

Table 3.1 Required data CPM for two-lane rural highways

Category	Data element	Value	Explanation
Horizontal Alignment	Type	Tangent/Curve/Spiral/Deflection	Deflection means a sudden change in heading not related to the curvature, it may happen on intersections or in the case that there is a very short curve that changes the heading of the highway but is not detected in surveillance
	Start Station		
	End Station		
	Curve Radius		
	Direction of Curve		
	Radius Position		Just for Spiral Type: before or after curves
	Deflection Angle		Just for Deflection Type, used on intersections or in the case that there is a very short curve that changes the heading of the highway but is not detected in surveillance
Vertical Alignment	Type	Vertical point intersection (VPI)/Tangent	
	VPI Station		For VPI
	Start Station		For Tangent

Category	Data element	Value	Explanation
	End Station		For Tangent
	Back Grade		Back means the portion of the vertical curve after PVI.
	Back Curve Length		Back means the portion of the vertical curve before PVI.
	Forward Grade		Forward means the portion of the vertical curve before PVI.
	Forward Curve Length		Forward means the portion of the vertical curve before PVI.
Lane Characteristics	Start Station		
	End Station		
	Side of Road	Both/Left/Right	Both means the element applies to both sides of the road. Left means it applies to the left side of the road when facing increasing stations. Right means it applies to the right side of the road when facing increasing stations
	Priority		When there are multiple lanes, lower value means closer to the centerline
	Type	Thru/Passing/Climb/ Left Turn/Right turn/Taper/Parking/ Bike	
	Start Width		
	End Width		
	Passing Prohibited on Opposing lane(s)	Yes/No	
Cross Slope	Station		
	Side of Road	Both/Left/ Right	Both means the element applies to both sides of the road. Left means it applies to the left side of the road when facing increasing stations. Right means it applies to the right side of the road when facing increasing stations
	Cross Slope		Could represent super elevation
Annual Average Daily Traffic	Start Station		
	End Station		
	Year		
	AADT		
Design Speed	Start Station		
	End Station		
	Speed		

Table 3.2 Optional data in CPM for two-lane rural highways

Category	Data element	Value	Explanation
Driveway Density	Start Station		
	End Station		
	Driveway Density (driveways/mi)		
Roadside Hazard Rating	Start and End Station		
	Roadside Hazard Rating	1 – 7	
Two-way Left Turn Lane	Start Station		
	Start Centerline Offset (ft)		
	Begin Full Width		
	Lane Width (ft)		
	End Full Width		
	End Centerline Offset (ft)		
	End Station		
Shoulder Section	Start Station		
	End Station		
	Side of Road	Both/Left/Right	Both means the element applies to both sides of the road. Left means it applies to the left side of the road when facing increasing stations. Right means it applies to the right side of the road when facing increasing stations
	Shoulder Side	Outside/Inside	
	Start Slope (%)		
	End Slope (%)		
	Start Width		
	End Width		
	Material	Paved/ Grave/ Turf/ Unknown	
	Rumble Strips	Yes/No	
	Priority		
Lighting	Start Station		
	End Station		
Automated Speed Enforcement	Start Station		
	End Station		
Centerline Rumble Strip	Start Station		
	End Station		

3.1.2 Data Sources and Data Elements

The SDDOT provided the research team with state highway inventory, local roads inventory including Bureau of Indian Affairs (BIA) roads, and crash data. All data sources are in GIS format. It is unclear if the BIA road network is complete in the SD local roads system. The NDDOT provided the research team with state highway inventory in GIS files.

After comparing inventories with the IHSDM data requirements; horizontal curve, vertical alignment and superelevation data were not found. Collecting vertical alignment and cross slope data, horizontal curve data requires field survey which is beyond the scope of the project. Instead, experimental data collection has been conducted at a small scale to test if horizontal curve information can be retrieved from the two-dimensional sources such as GIS shapefiles, aerial photo images, etc.

In HSM, it is required that the calibration sites be randomly selected from a larger set of candidate sites to avoid the bias caused by including only sites with either high or low crash frequencies. Therefore, the calibration sites could be first randomly selected from the road set of interest, and then the data collection of curves will be conducted on those roadways.

3.1.3 Data Collection Methods

3.1.3.1 GPS

Curve information may be retrieved from the geographic coordinates collected by GPS device. Figure 3.1 shows the use of Civil 3D to import transformed GPS data. After that, the points along the curve in Civil 3D can be utilized to calculate the curve parameters.

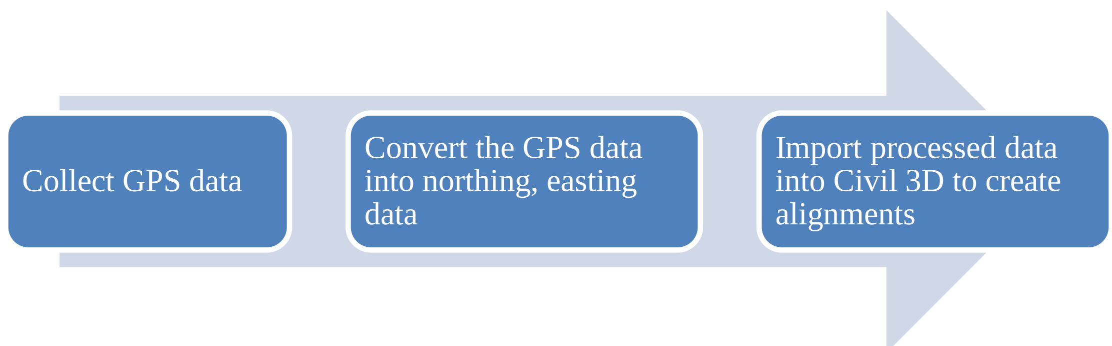

Figure 3.1 GPS data process

In an example with two curves on United States Highway 14, the curves retrieved in Civil 3D were compared to the curves in Google Earth.

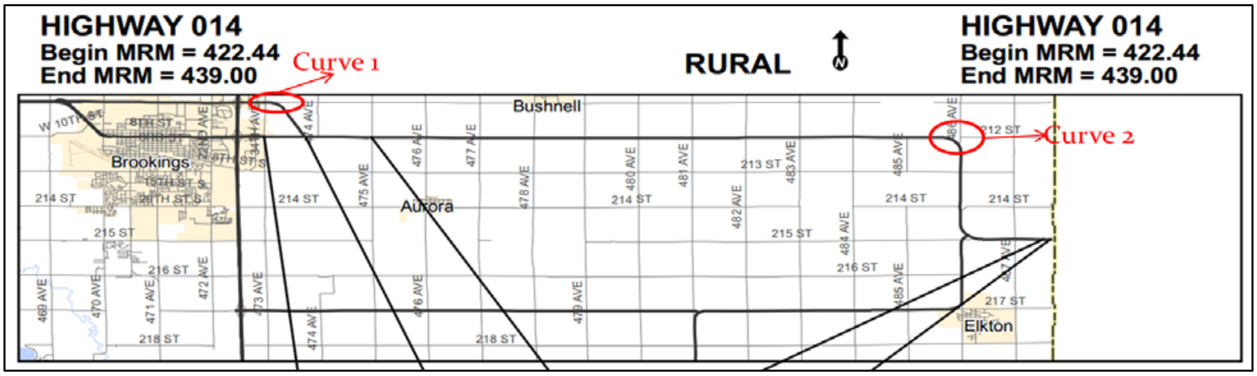

Figure 3.2 Curves on Highway 14

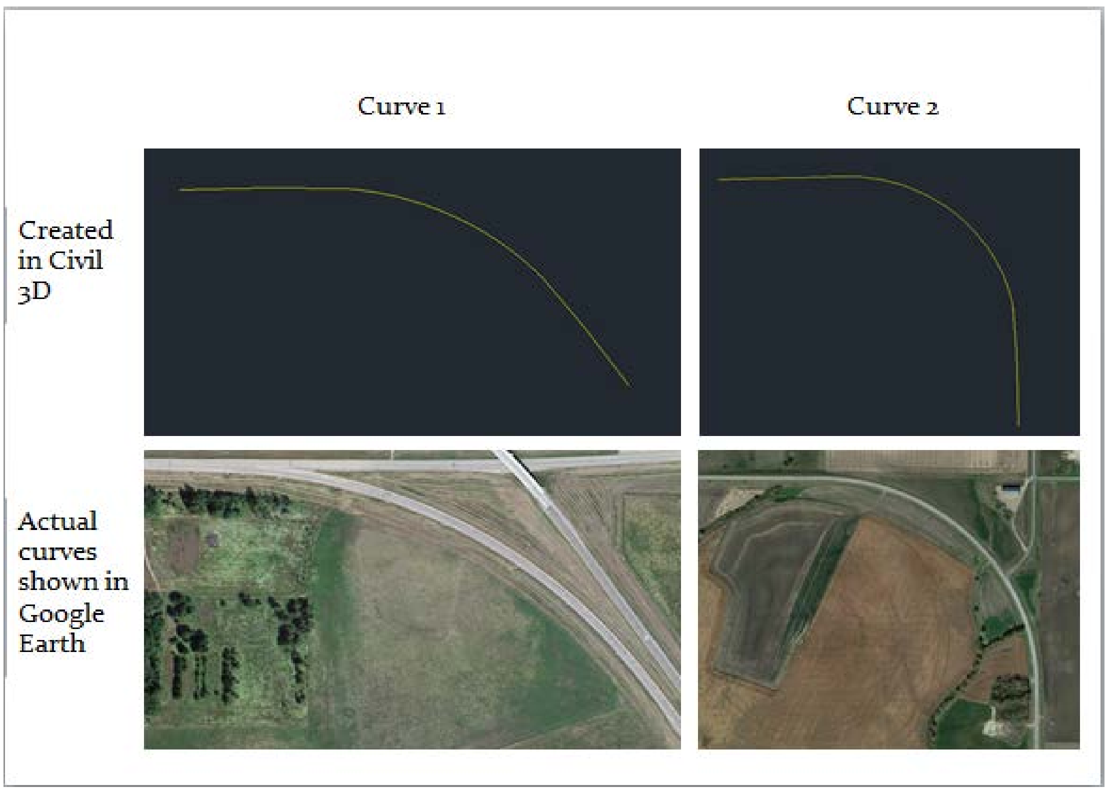

Figure 3.3 Comparison between curves created in Civil 3D and Google Earth

3.1.3.2 GIS Shapefile

In the shapefile of the roadways, roadways are represented by non-linear lines. As Figure 3.4 shows, the highlighted line may include three tangents and two curves. Curve data from the shapefile can be retrieved by separating the line into equal interval points.

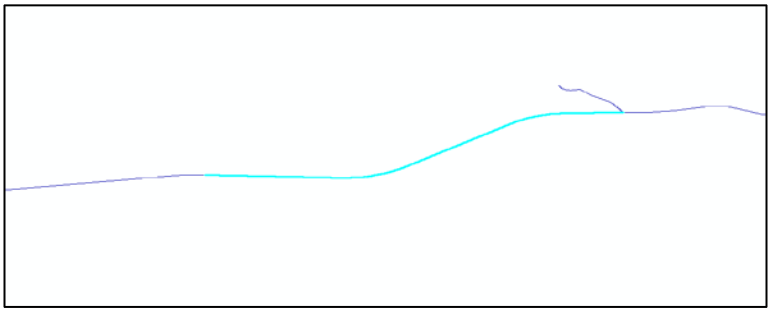

Figure 3.4 Roadways in the shapefile

3.1.3.3 Imagery Data

Imagery data can be accessed from sources such as Google Earth, Bing Maps, and so on. Google Earth was chosen as the source for its compatibility with Civil 3D.The process is shown in Figure 3.5.

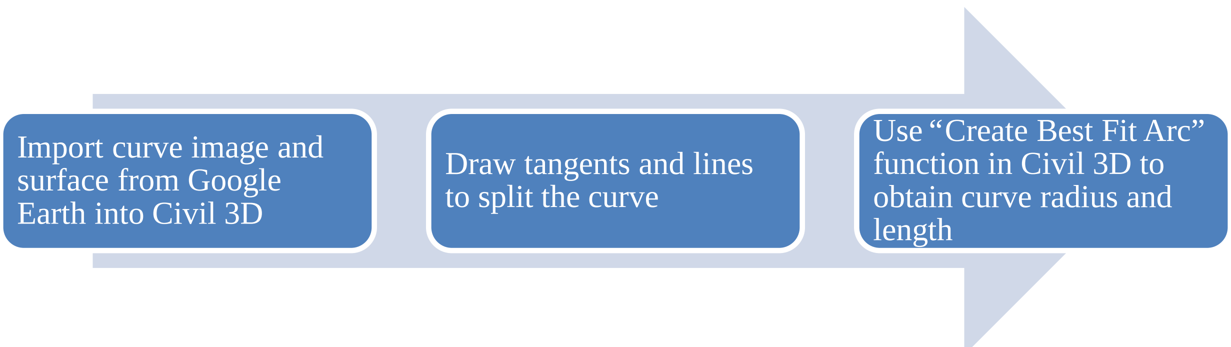

Figure 3.5 Imagery data process

Figure 3.6 shows how a curve image is processed in Civil 3D where the dashed red line represents the simulated curve in Civil 3D and the curve information is displayed in Figure 3.7.

Figure 3.6 Curve from Google Earth to Civil 3D

Property	Value
Entity1	Arc
Length	317.70'
Radius	321.29'
Start Po...	(27849.5308',36...
End Point	(27802.4129',36...
Delta A...	56.6555 (d)
Chord ...	N8° 53' 22"W
Start Dir...	N19° 26' 18"E
End Dir...	N37° 13' 02"W

Figure 3.7 Generating interface

3.1.3.4 Summary

Among the three methods, the GPS method would have the highest level of accuracy if properly applied because it collects the data directly from the field, but it also requires the most formidable work. The other two methods need less effort to get data, but data accuracy cannot be guaranteed. The comparison provides a fair assessment of three approaches to extracting horizontal curve data from different data sources with the clear trade-offs among accuracy, level of effort, and costs. Despite a promising application, it was decided to hold off on a large-scale curve data collection till the completion of the evaluation with any available data elements maintained by the transportation agencies (e.g. NDDOT, SDDOT). Future efforts will be taken to collect data mainly by GPS devices, and some studies will also be conducted to improve the precision of these two methods so that the three methods can be combined to optimize the process of data collection.

3.1.4 Data Processing

Local roads are roads that are primarily used to gain access to the property bordering them. These roads have the lowest speed limit and carry low volumes of traffic. In some areas, these roads may be unpaved. Tribal roads are those roads located on Indian Reservations and within tribal communities and are maintained by BIA Region offices. These roads serve as major access points for providing services to tribal communities.

For South Dakota, in the attribute table of the shapefile is a field called FED_DOMAIN indicating Federal ownership. Codes 1 and 5 represent BIA and Tribal roads, respectively. In this way, roads having these two codes in that field were considered as tribal roads. Local roads were identified based on the field indicating the functional classification of the road. Roads coded as “rural local roads” were identified as local roads.

For North Dakota, tribal roads are not separated from other roads. To identify them, the shapefile of non-federal roads was laid on top of the shapefile indicating tribal regions. Roads located in the tribal regions were considered as tribal roads. Local roads in North Dakota are those with the function class field as “local.”

3.2 Crash Data Collection

3.2.1 Data Requirement and Data Source

Crash data is mainly used in the calibration procedure, and is used to evaluate the crash prediction module as well. Data required for calibration in IHSDM are listed in Table 3.3. Crash data information was requested from the SDDOT, but crash data was not available from the NDDOT. Due to the lack of crash data from North Dakota, only SD roadway sections were calibrated in this study. The calibration methodologies and procedures developed from the SD data can be conveniently transferred to ND once crash data becomes available.

Table 3.3 Data requirement for calibration

Category	Data Element	Value	Explanation
Required Site Data	Length (mi)		
	Left Lane Width (ft)		
	Right Lane Width (ft)		
	Left Paved Shoulder (ft)		
	Right Paved Shoulder (ft)		
	Left Gravel Shoulder (ft)		
	Right Gravel Shoulder (ft)		
	Left Turf Shoulder (ft)		
	Right Turf Shoulder (ft)		
	Curve Radius (ft)		
	Curve Length (mi)		
	TWLTL	Yes/No	The presence of a TWLT lane
Required Crash/ Traffic Data	Years of Crash Data	1 or 2 or 3	Number of years of crash data for the site
	Year 1		The first year of AADT data
	Year 1 AADT		
	Year 2		The second year of AADT data
	Year 2 AADT		
	Year 3		The third year of AADT data
	Year 3 AADT		
	Observed Crashes		Total number of crashes observed at the site during the specified years

3.2.2 Data Processing

To apply the crash data, the number of crashes on each road segment should be known. However, the crash data appears as individual points in GIS while the roadways appear as lines. One method to count the number of crashes on each segment is through "Spatial Join," which is a function in GIS. Spatial join can automatically count the number of the points within a certain distance of a line, and by properly specifying the value of the distance (so-called buffer distance) the number of crashes that occurred on the roadways is obtained. The buffer distance in this project is set as 30 ft. which is long enough to guarantee each crash will be joined to one segment. A resulting problem is that some crashes may be joined to several nearby segments, so a manual check has been made to rule out all mistakenly joined crashes by comparing the location description of crashes and joined segments.

3.3 Data Description

3.3.1 Rural Local Roads

Through road and crash data collection, 657 rural local segments were included on which 91 crashes happened.

Figure 3.8 shows the histogram of segment lengths for all 657 segments. The majority of segments have lengths around 1 mi, and the mean segment length is 1.198 mi.

Figure 3.9 is a histogram of lane width and Figure 3.10 is a histogram of shoulder width. The base conditions of the HSM SPF are 12-ft lane widths and 6-ft shoulder widths. Among 657 segments, 234 segments have 12-ft lanes and only two segments have 6-ft wide shoulders. Segments with 12-ft lanes are the most representative ones, while more than 90% of the segments don't have shoulders. The mean lane and shoulder widths are 10.8 ft and 0.12 ft, respectively.

Figure 3.11 is a histogram of an average of three-year AADT. This shows that 535 out of 657 segments have traffic volumes lower than 200 veh/day, which is considered as a very low volume. The mean average AADT is 132 veh/day.

Figure 3.12 shows the histogram of crash counts in three years. This shows that 577 out of 657 segments experienced no crashes in these three years, and only one segment experienced three crashes. In general, the crash level for studied segments is very low. The mean crash frequency in three years is 0.14.

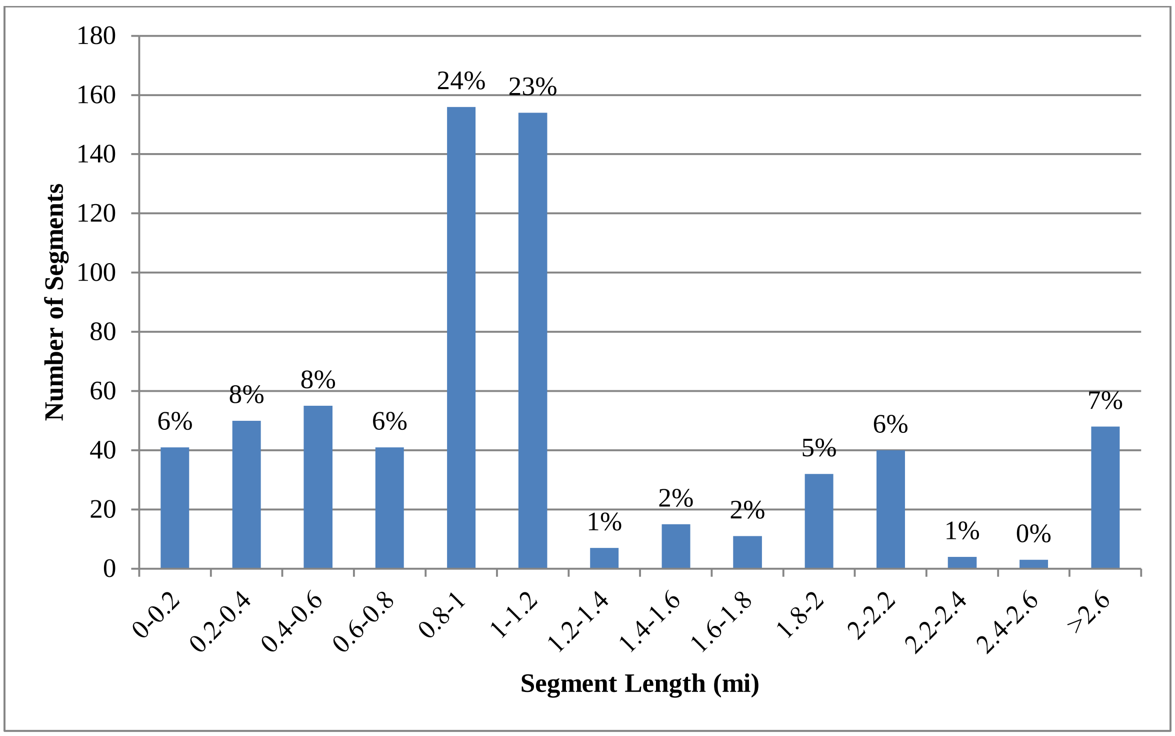

Figure 3.8 Histogram of segment length

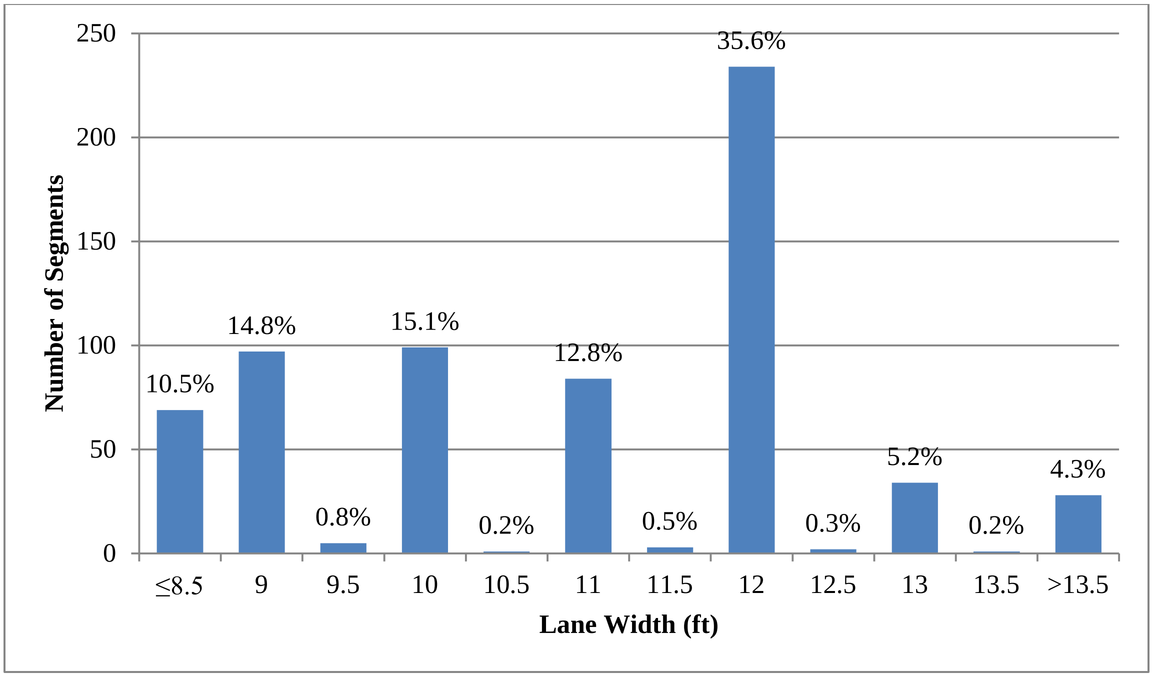

Figure 3.9 Histogram of lane width

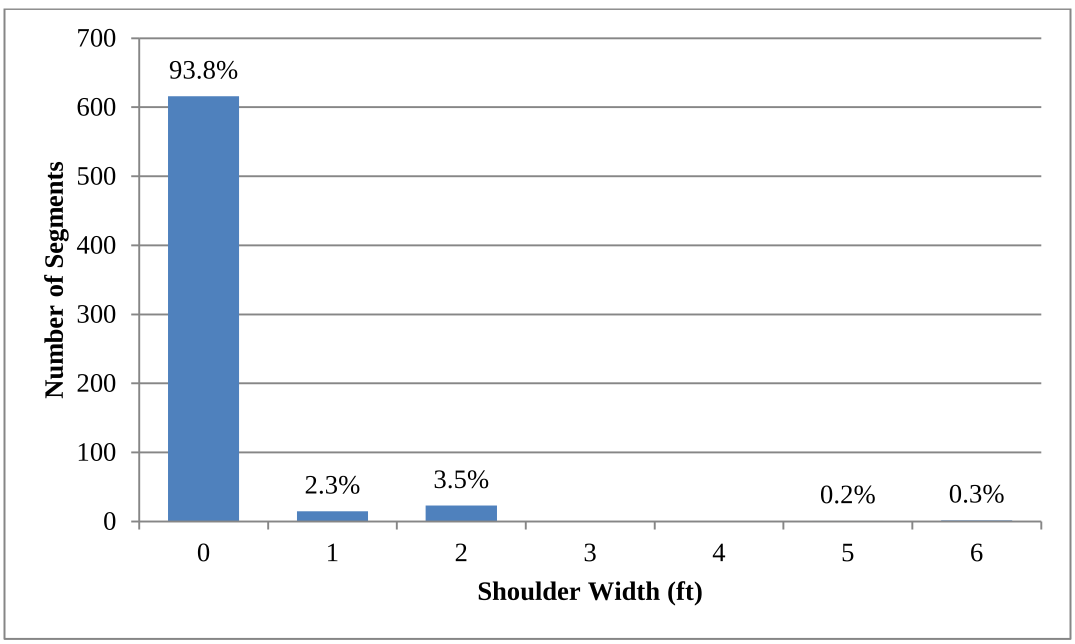

Figure 3.10 Histogram of shoulder width

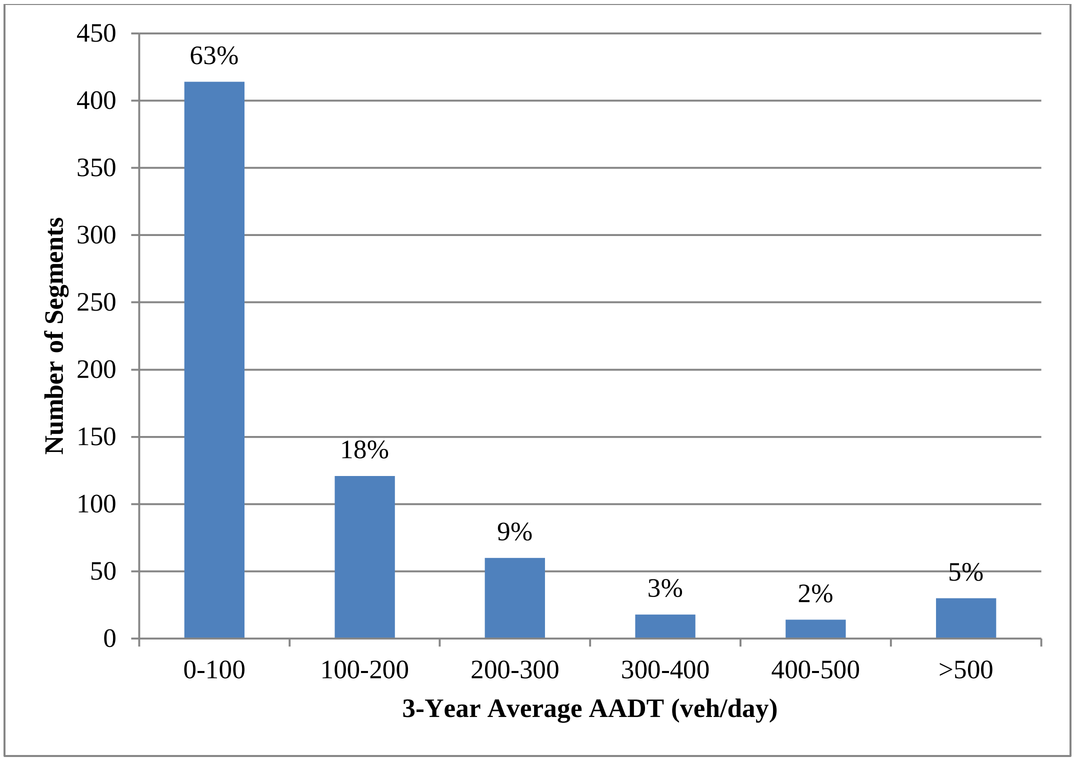

Figure 3.11 Histogram of average AADT over three years

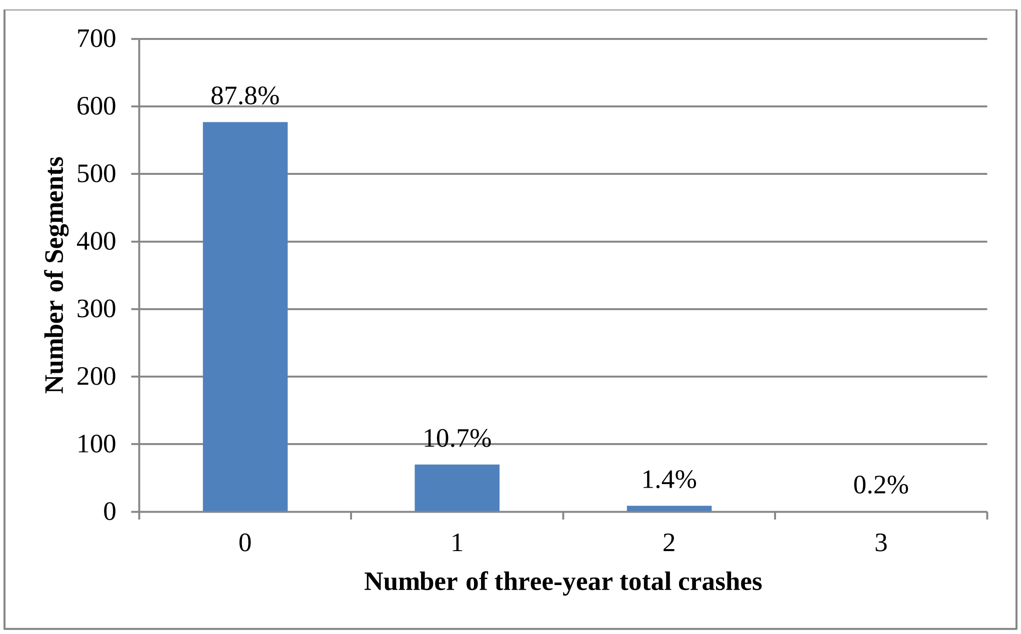

Figure 3.12 Histogram of number of three-year total crashes

3.3.2 Tribal Rural Roads

A total of 56 tribal rural roads were included in the study on which 41 crashes took place.

Figure 3.13 is a histogram of segment length. Note there are no segments with lengths smaller than 0.3 miles. The mean segment length is 3.56 mi.

Figure 3.14 and Figure 3.15 are histograms of lane widths and shoulder widths, respectively. Among 56 segments, 26 segments have 12-ft lanes, holding the largest the proportion. Only two segments have 6-ft shoulders, while 44 of them have no shoulders. The mean lane widths and shoulder widths are 12.9 ft and 0.63 ft.

Figure 3.16 shows the histogram of average AADT over three years. In general, most segments have either very low or relatively high traffic volumes. 45% of segments have AADT less than 200 veh/day, while 29% of them have volumes greater than 800 veh/day. The mean average AADT is 678 veh/day.

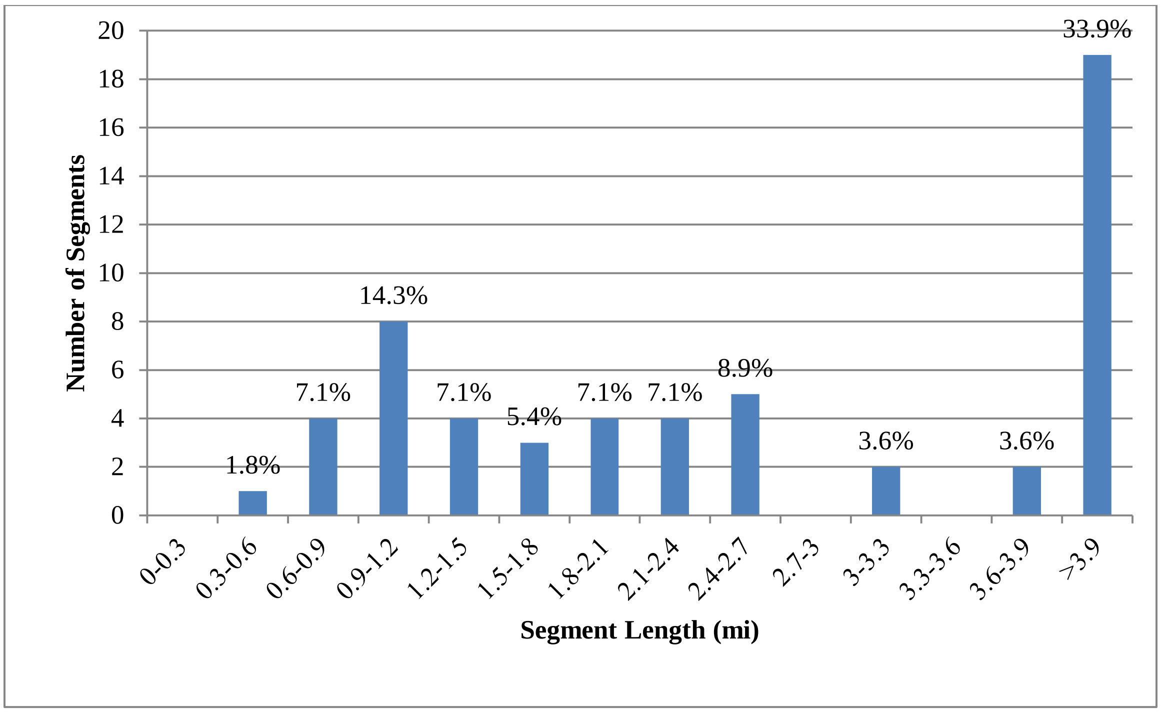

Figure 3.13 Histogram of segment length

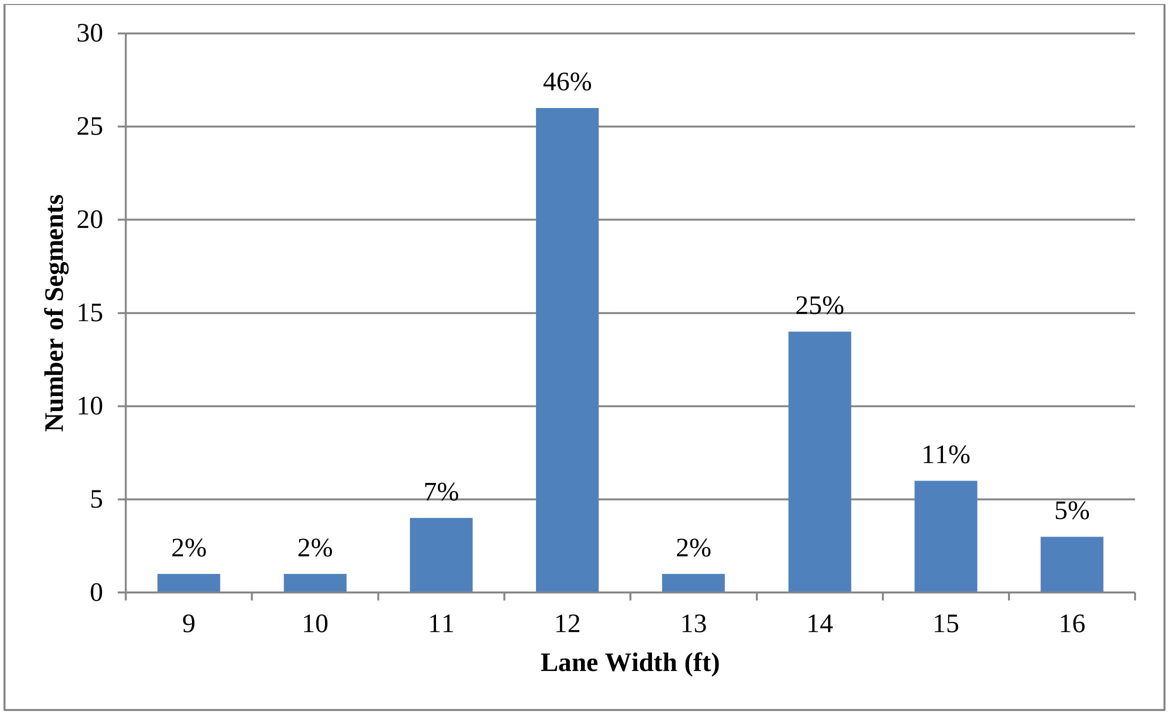

Figure 3.14 Histogram of lane width

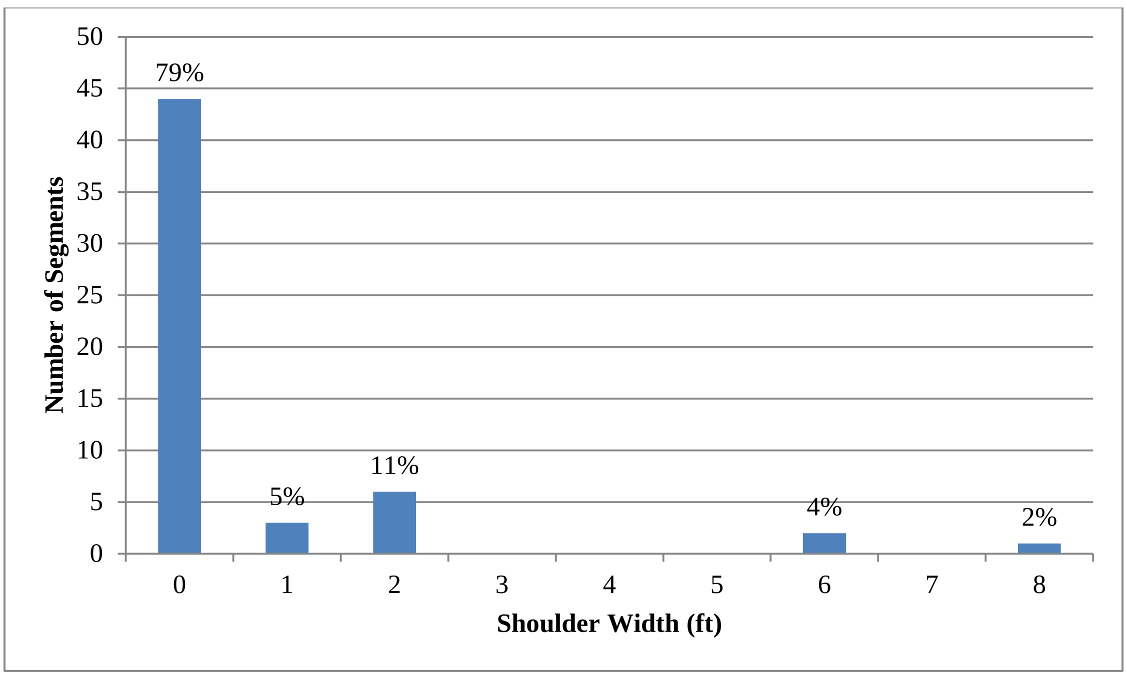

Figure 3.15 Histogram of shoulder width

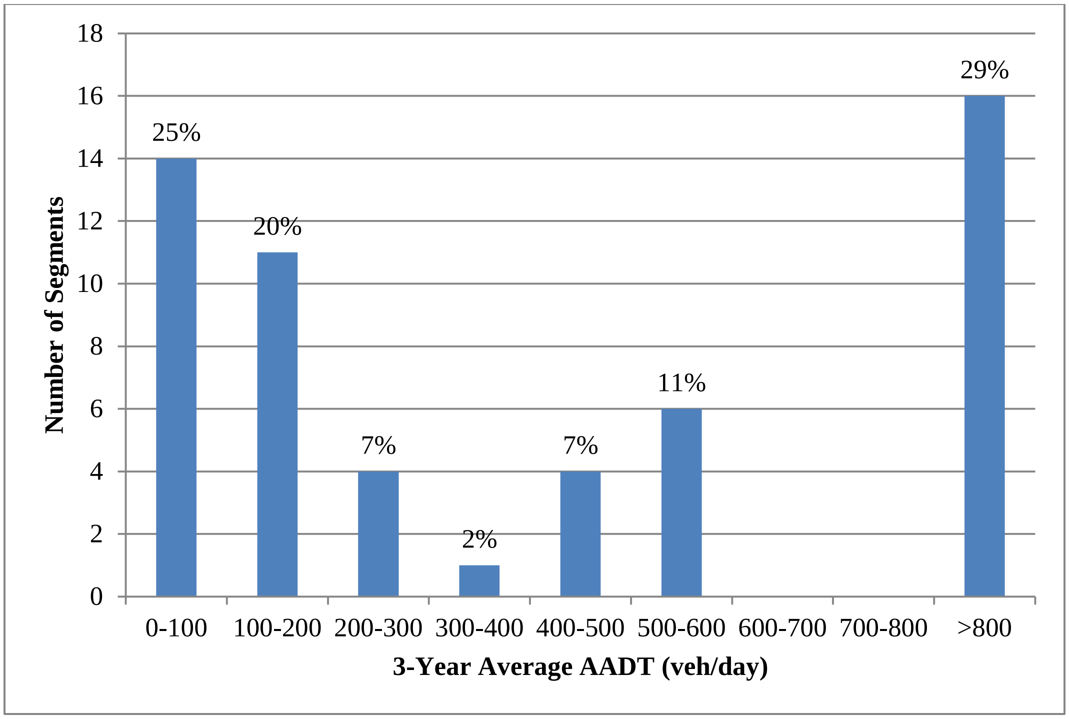

Figure 3.16 Histogram of average AADT over three years

Figure 3.17 is a histogram of three-year combined crash frequency. The range is from 0 to 4. And it shows most segments experienced no crashes in three years, and very few segments experienced four crashes.

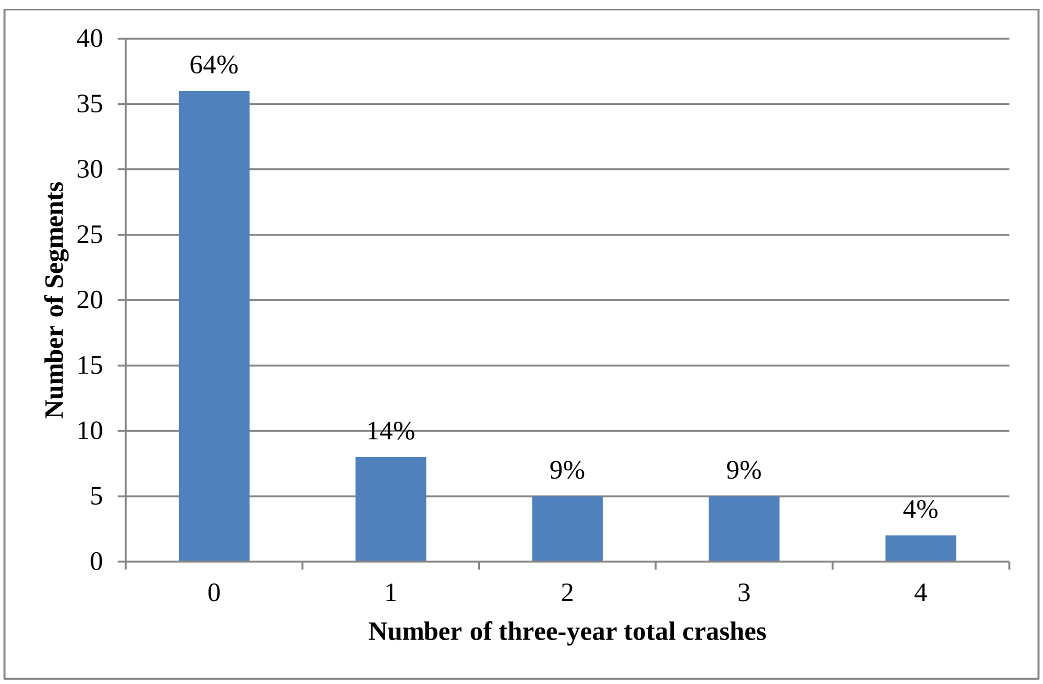

Figure 3.17 Histogram of number of three-year total crashes

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4. IHSDM TESTING AND APPLICATION

To run the crash prediction module in IHSDM, data should be imported first as Figure 4.1 shows. Data can be entered manually or through a customized import module developed in Microsoft Excel.

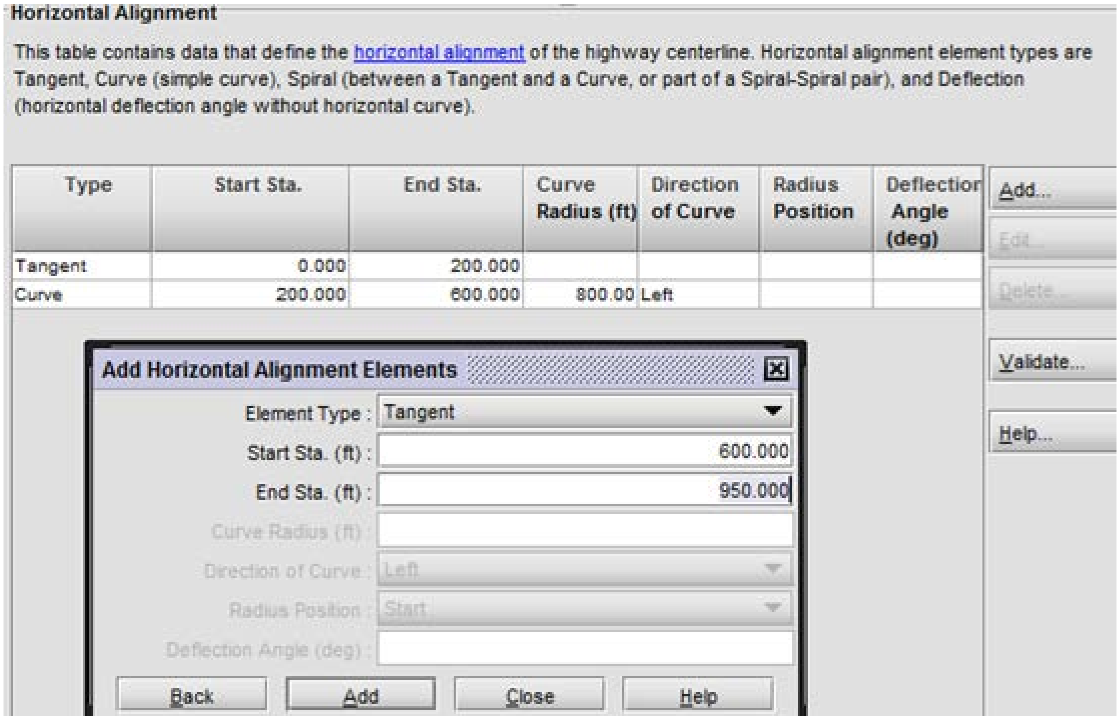

Figure 4.1 Data input interface in IHSDM

IHSDM was tested with a 4-mile long roadway section (USH 83 in Custer, SD). The raw data was retrieved from the SDDOT Roadway Inventory System (RIS). RIS has detailed roadway geometric design data by station including horizontal alignment, deflection angle, curve degree, start and end stations of the curve, curve radius, etc. The import module was developed to process the original data as Figure 4.2 shows the original data and Figure 4.3 shows the data after applying the import module.

Highway Nbr	MRMNbr	Displacement Nbr	Curve Direction	Deflection Angle Degrees	DeflectionAngleMinutes	Deflection Angle Seconds	Curve Degrees	Curve Minutes	Curve Seconds	Spiral LengthIn	Spiral Length Out	Design Speed
087	48.00	0.127	L	11	45	0.00	2	0	0.00	0	0	75
087	48.00	0.276	L	4	50	0.00	5	0	0.00	0	0	55
087	48.00	0.395	L	18	30	0.00	10	0	0.00	0	0	40
087	48.00	0.543	R	50	15	0.00	15	0	0.00	0	0	35
087	48.00	0.610	L	30	0	0.00	20	0	0.00	0	0	30
087	48.00	0.702	R	16	0	0.00	10	0	0.00	0	0	40
087	48.00	0.777	R	13	0	0.00	5	0	0.00	0	0	55
087	48.00	0.886	R	5	21	0.00	2	0	0.00	0	0	75
087	49.00	0.075	L	34	12	0.00	15	0	0.00	0	0	35
087	49.00	0.193	R	36	49	0.00	16	0	0.00	0	0	30

Figure 4.2 Original data of horizontal alignment

Type	Start Sta.	End Sta.	Curve Radius (ft)	Direction of Curve	Radius Position	Deflectior Angle (deg)
Tangent	0.000	670.560				
Select ...	670.560	1+258.060	2,864.79	Left		
Tangent	1+258.060	1+457.280				
Curve	1+457.280	1+553.947	1,145.92	Left		
Tangent	1+553.947	2+085.600				
Curve	2+085.600	2+270.600	572.96	Left		
Tangent	2+270.600	2+867.040				
Curve	2+867.040	3+202.040	381.97	Right		
Tangent	3+202.040	3+220.800				
Curve	3+220.800	3+370.800	286.48	Left		
Tangent	3+370.800	3+706.560				
Curve	3+706.560	3+866.560	572.96	Right		
Tangent	3+866.560	4+102.560				
Curve	4+102.560	4+362.560	1,145.92	Right		
Tangent	4+362.560	4+678.080				
Curve	4+678.080	4+945.580	2,864.79	Right		
Tangent	4+945.580	5+676.000				
Curve	5+676.000	5+904.000	381.97	Left		

Figure 4.3 Input interface with the formatted data

After entering all required data, the Policy Review Module (PRM) and Crash Prediction Module (CPM) can generate the reports. PRM checks a design relative to the range of values for critical dimensions recommended in the AASHTO design policy and CPM provides estimates of expected crash frequency and severity.

The PRM generates a series of tables and figures to compare the actual values of elements to those required by the policy, and comments on whether the design complies with the policy standard are also provided. Figure 4.4 illustrates a portion of the table for radius of the curve which displays the road radius and policy radius, road speed and effective design speed, and a corresponding comment for each segment. Whether the design satisfies the standard is distinguished by different colors along with the comments. As shown in Figure 4.4, rows in white indicate the segments meet the design criteria while rows in red imply the corresponding segments that have failed to fulfill the requirements.

Start Location	End Location	Road Radius (ft)	Policy Radius (ft)	Effective Design Speed (mph)	Speed (mph)	Functional Class	Surface Type	Comment
670.560	1+258.060	2,864.79	587.00	> 80	45	Arterial		Road value is within controlling criteria
1+457.280	1+553.947	1,145.92	587.00	58	45	Arterial		Road value is within controlling criteria
2+085.600	2+270.600	572.96	587.00	44	45	Arterial		Road value varies from controlling criteria
2+867.040	3+202.040	381.97	587.00	37	45	Arterial		Road value varies from controlling criteria
3+220.800	3+370.800	286.48	587.00	34	45	Arterial		Road value varies from controlling criteria
3+706.560	3+866.560	572.96	587.00	44	45	Arterial		Road value varies from controlling criteria
4+102.560	4+362.560	1,145.92	587.00	58	45	Arterial		Road value is within controlling criteria
4+678.080	4+945.580	2,864.79	587.00	> 80	45	Arterial		Road value is within controlling criteria
5+676.000	5+904.000	381.97	587.00	37	45	Arterial		Road value varies from controlling criteria
6+299.040	6+529.144	358.10	587.00	36	45	Arterial		Road value varies from controlling criteria
6+980.160	7+200.716	477.46	587.00	40	45	Arterial		Road value varies from controlling criteria
7+386.720	7+614.220	572.96	587.00	44	45	Arterial		Road value varies from controlling criteria
8+321.280	8+529.891	477.46	587.00	40	45	Arterial		Road value varies from controlling criteria

Figure 4.4 Part of the table of curve radius

In addition to the tables, reports also present figures to provide graphical information. Figure 4.5 depicts the trend lines of the K value, curve degree, curve radius, and available sight distances for both directions showing the change with the increase of station. Required sight distances are compared with available values to provide more explicit information than tables only consisting of numeric values.

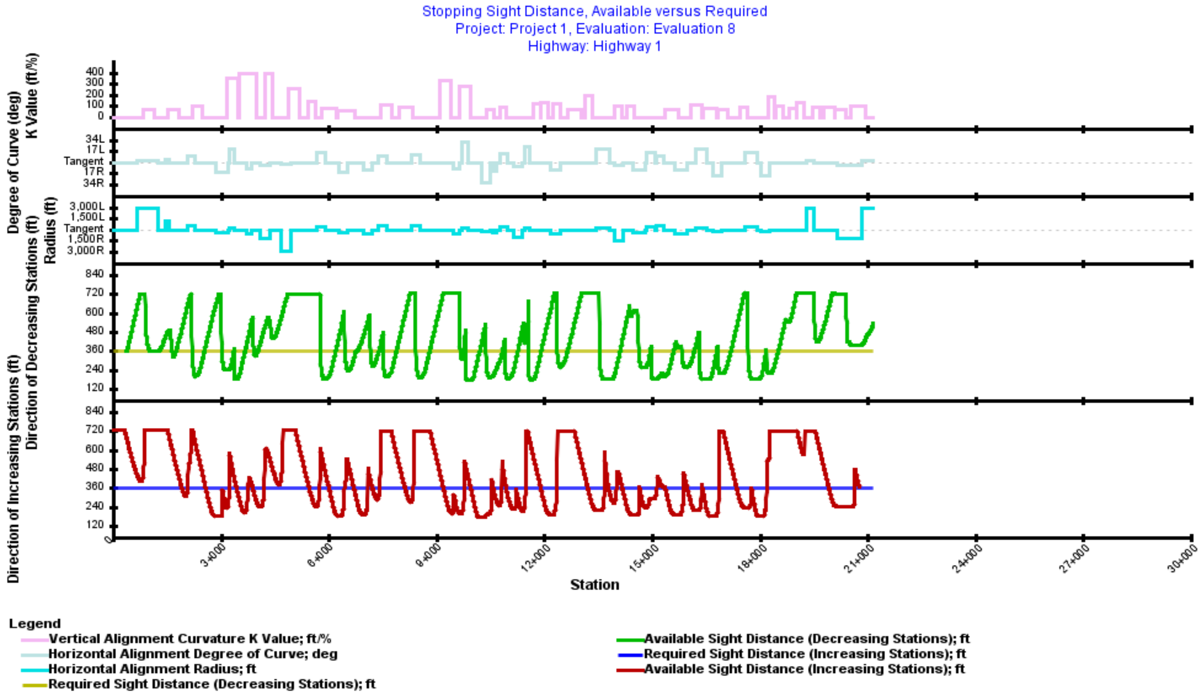

Figure 4.5 Figure for stopping sight distance

The CPM generates several tables of predicted crash frequencies and rates of the whole roadways, by highway segments, and by horizontal design elements such as Figure 4.6. The expected crash frequencies and rates can be compared with the observed data to evaluate the appropriateness of the module or be used to identify the sites with high risk.

Start Location	End Location	Length (mi)	Expected No. Crashes for Evaluation Period	Crash Rate (crashes/mi/yr)	Travel Crash Rate (crashes/million veh-mi)
0.000	670.560	0.1270	0.047	0.0622	0.85
670.560	987.360	0.0600	0.030	0.0839	1.15
987.360	1+258.060	0.0513	0.024	0.0763	1.04
1+258.060	1+457.280	0.0377	0.013	0.0565	0.77
1+457.280	1+553.947	0.0183	0.026	0.2394	3.28
1+553.947	1+679.040	0.0237	0.008	0.0565	0.77
1+679.040	2+085.600	0.0770	0.029	0.0622	0.85
2+085.600	2+270.600	0.0350	0.061	0.2890	3.96
2+270.600	2+344.320	0.0140	0.005	0.0622	0.85
2+344.320	2+867.040	0.0990	0.034	0.0565	0.77
2+867.040	3+202.040	0.0634	0.088	0.2303	3.15
3+202.040	3+220.800	0.0036	0.001	0.0565	0.77
3+220.800	3+326.400	0.0200	0.065	0.5404	7.40
3+326.400	3+370.800	0.0084	0.027	0.5404	7.40
3+370.800	3+706.560	0.0636	0.022	0.0565	0.77

Figure 4.6 Part of table of expected crash frequencies and rates by highway segment

Similar to the PRM, Figure 4.7 illustrates crash rate changes along the roadway.

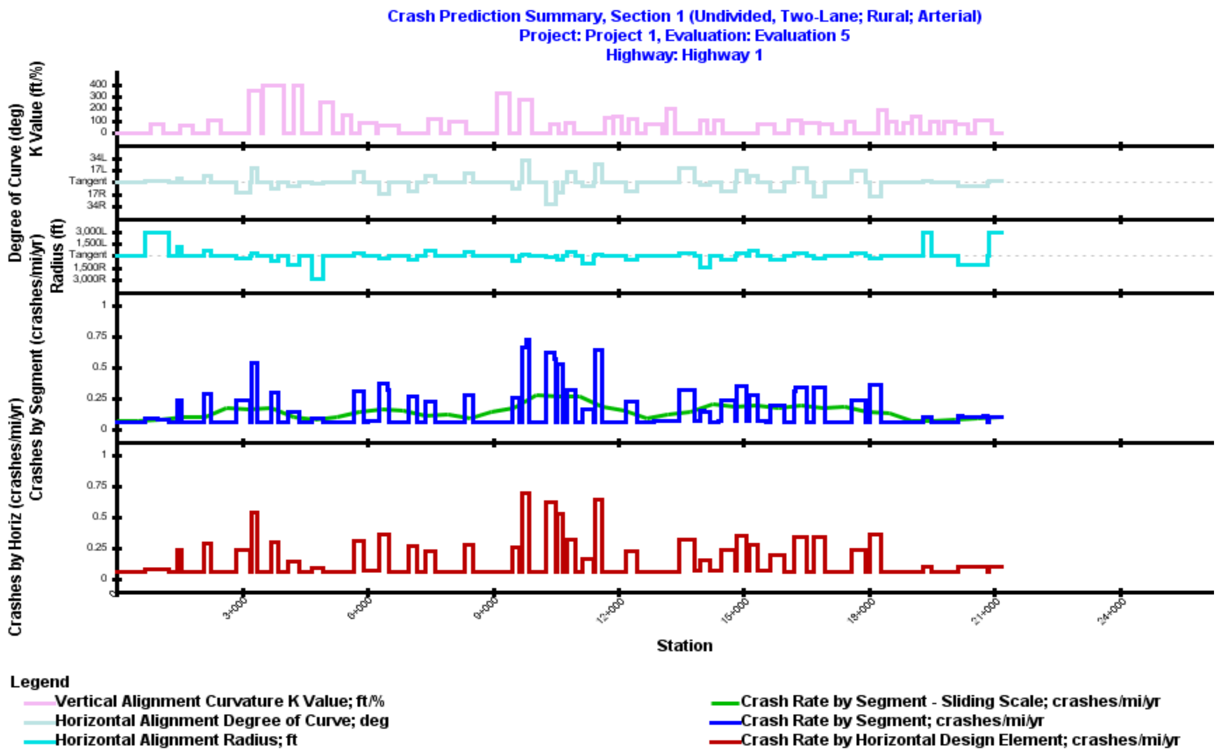

Figure 4.7 Crash prediction summary

The PRM along with CPM can identify the sites with design components not meeting the standards or with high crash risks. Planned remedial measurements such as widening the lane surface and increasing the radius of the curve can also be evaluated through these two modules before being applied. For construction plans, two modules can be employed to check whether the plans meet the policy requirements and compare alternative plans from the safety perspective.

5. SAFETY MODULES FOR RURAL LOCAL AND TRIBAL ROADS

It becomes more attainable to establish performance-based safety goals with the Highway Safety Manual (HSM). However, the safety performance functions (SPFs) in the HSM may not be accurate because they are not calibrated to local conditions. In addition, each SPF and crash modification factor (CMF) assumes a set of base site conditions which may not be realistic for local roadways.

Although calibration procedures are available in HSM Part C Appendix A, they should be refined or modified to accommodate local data availability and roadway, traffic, and crash characteristics. It is also necessary to determine a set of base conditions applicable to local highways.

The following calibration process includes establishing new base conditions, developing SPFs, converting CMFs to base conditions as well as substituting default values with state-specific values.

5.1 Modules for Rural Local Roads

Through the data collection, 657 roadway segments constituting more than 750 miles of roadways were included in the study, and 91 crashes were found to happen on these segments from 2009 to 2011. Table 5.1 presents the descriptive statistics of numeric independent variables. Note that AADT is the three year average of AADT from 2009 and 2011. Since only 2011 AADT was available in the road inventory, the growth factors for each county in SD were applied to generate AADT in 2009 and 2010, respectively.[23]

Table 5.1 Descriptive statistics of data variables.

Variable	Mean	Minimum	Maximum	Median	Standard Deviation
Segment Length (mi)	1.198	0.024	8.843	1	1.021
AADT (vpd)	132.7	1	2460.2	60	215.7
Lane Width (ft)	10.8	8	16	11	1.6
Shoulder Width (ft)	0.12	0	6	0	0.54
Speed Limit (mph)	52.2	15	65	0	7.7

Among 657 roadway segments, only one site has the identical HSM base conditions defined for rural two-lane two-way roads. The HSM suggests that jurisdiction-specific base conditions be designed by agencies to represent the most common characteristics of roads.[9] After reviewing all the roadway geometric characteristics, the new base conditions for SD rural local two-lane two-way roads are as follows:

- Lane width (LW): 12 ft.
- Shoulder width (SW):None
- Shoulder type: Paved
- Roadside hazard rating (RHR): 3
- Driveway density (DD): 5 driveways per mile
- Horizontal curvature: None
- Vertical curvature: None
- Centerline rumble strips: None
- Passing lanes: None

- Two-way left-turn lanes: None
- Lighting: None
- Automated speed enforcement: None
- Grade Level: 0%

In total, 138 segments satisfy the new base conditions, and 26 crashes happened on these segments. Data collected from the rural local roads defined and established the most representative roadway type among all local rural two-lane two-way highways in SD. The selection of suitable base conditions for SD highways will lay the foundation for the SD SPFs.

The HSM calibration process in Part C Predictive Methods Appendix A was reviewed and refined. Figure 5.1 displays the HSM calibration procedures (in hollow textboxes), along with proposed changes (in shaded textboxes) to illustrate where they might occur. The modifications include: a) develop new SPF models for new base conditions; b) convert CMFs to new base conditions; c) substitute default values with state-specific values; and d) determine region-specific calibration factors. It is anticipated that the SPFs developed with SD data will be more accurate and more reliable for predicting crash frequencies under base conditions.

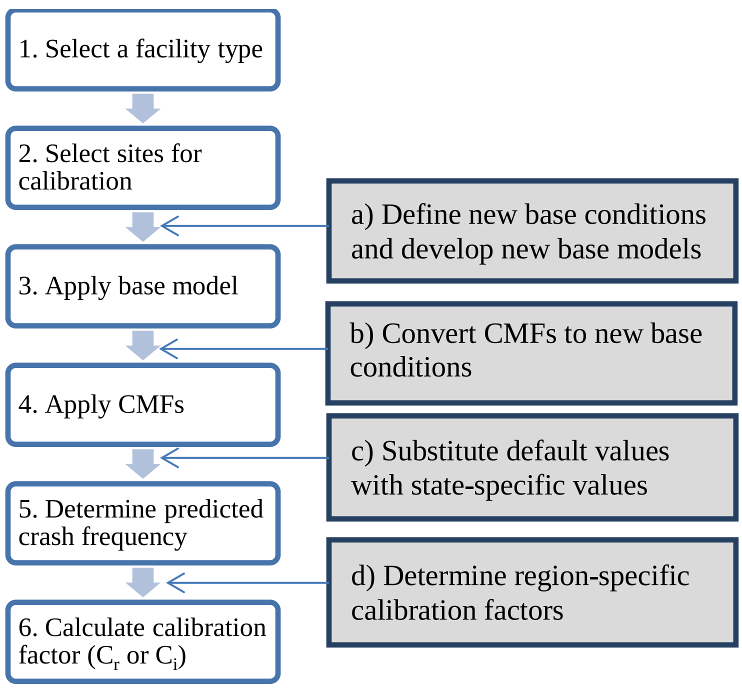

Figure 5.1 Proposed modifications to calibrate HSM Part C predictive methods

5.1.1 Safety Performance Functions (SPFs)

The SPFs in the current HSM are in a very simple form for roadway segments: crash frequency is the function of AADT and segment length is always assumed to be proportional to crash frequency. In South Dakota, from 2009 to 2011, 91 crashes were reported on 657 roadway segments. Using IHSDM which is a faithful implementation of HSM Part C, the predicted crash frequency was 59.2. The calibration factor was computed as 1.537. The calibrated HSM SPF without any modifications is shown in Equation 2.

$$N_{SD\ Local} = 1.537 \times AADT \times L \times 365 \times 10^{-6} \times e^{(-0.312)} = 4.111 \times 10^{-4} \times AADT \times L \quad (2)$$

5.1.1.1 Development of SPFs Using Data Conforming to New Base Conditions

Two methods are used for developing SD SPFs. The first method uses only data that conforms to base conditions. In this approach, the SPFs were developed from a group of homogenous sites, and the CMFs are completely independent of SPFs. This method may be limited by insufficient data to develop statistically meaningful models, and may require extrapolation as some values may exceed the range of data used.

Two negative binomial models, Models 1 and 2, were developed. In both models, segment length was treated as an offset rather than a variable because crash frequency on a segment is considered to be proportional to the segment length.[9] The model results are shown in Table 5.2.

Table 5.2 Parameter estimate for model 1 and 2

	Model 1 ($N = \beta_0 \times AADT^{\beta_1} \times L$)		Model 2 ($N = \beta_0 \times AADT \times L$)	
Parameter	Estimate	p-Value	Estimate	p-Value
Intercept [ln(β0)]	-6.2357	<.0001	-7.6109	<.0001
Log(AADT) [β1]	0.7363	0.0012	–	–
Overdispersion	1.20		1.17	
Formula	$N = AADT^{0.7363} \times L \times 1.958 \times 10^{-3}$		$N = AADT \times L \times 4.950 \times 10^{-4}$	

5.1.1.2 Development of Jurisdiction-specific SPFs using the Whole Data Set

The second approach is to develop a full model using data with a broader spectrum of conditions than the base conditions, and then set the values of variables to the base conditions. While this method takes advantage of all available sites and their characteristics, it invites unobserved factors that may bias the estimates. Moreover, the variables may interact with each other and jointly affect crash frequencies, which cannot be effectively modeled.

A statistical analysis was performed to develop SPFs from the entire data set. Variables include AADT, segment length, lane width, surface type, shoulder width, shoulder type, terrain, and speed limit. Only AADT and segment length could produce logical and reasonable results despite several attempts at data transformation and creating a segment length indicator variable (assumed to represent unobserved factors like passing lane). Models with additional factors were rejected because of the poor goodness-of-fit and unreasonable signs for independent variables. Due to the low sample means, data overdispersion cannot be accurately captured by the negative binomial regression model.[24] The Poisson regression models were developed, and the results are shown in Table 5.3.

Table 5.3 Parameter estimate for model 3 and 4

	Model 3 ($N = \beta_0 \times AADT^{\beta_1} \times L$)		Model 4 ($N = \beta_0 \times AADT \times L$)	
Parameter	Estimate	p-Value	Estimate	p-Value
Intercept [ln(β_0)]	-7.5833	<.0001	-7.7621	<.0001
Log(AADT) [β_1]	0.9652	<.0001	–	–
Formula	$N = AADT^{0.9652} \times L \times 5.089 \times 10^{-4}$		$N = AADT \times L \times 4.256 \times 10^{-4}$	

5.1.2 CMF Calibration

After defining and establishing the new base conditions, CMFs must be converted accordingly, as they represent quantitative changes in predicted crash frequencies resulting from site characteristic variations from base conditions. If the CMF value is a scale factor, the new base condition will be reset to one and the others can be adjusted by a corresponding multiplier. If the CMF value is a function of AADT, the relationship will be regressed under the new base conditions.

5.1.2.1 Conversion of CMFs

For rural local two-lane two-way highways, the only difference between changed base conditions and HSM base conditions was shoulder width. The HSM CMFs for shoulder width is shown in Table 5.4. No shoulder width became the base. The purpose of converting CMFs was to compute the ratio of CMFs for non-zero shoulder width and CMFs for zero shoulder width. For example, the ratio of 1.00 and 1.1, which is 0.91, was the adjusted CMF for 6-ft. shoulder width when AADT < 400. Since the HSM CMF for 0 ft. is a formula of AADT when AADT is between 400 and 2000, a single value for adjusted CMFs at other levels of shoulder width cannot be generated. A new linear regression model is needed for each non-scale factor CMF. Figure 5.2 shows the formula of CMF for 6-ft. shoulder widths when AADT is between 400 and 2000. The regression process was repeated for other non-zero shoulder widths, and the results are listed in Table 5.4.

Table 5.4 CMF for shoulder width

		AADT		
	Shoulder Width	<400	400 to 2000	>2000
HSM CMF*	0 ft.	1.1	$1.10+2.5\times10^{-4}$(AADT-400)	1.5
	2 ft.	1.07	$1.07+1.43\times10^{-4}$(AADT-400)	1.3
	4 ft.	1.02	$1.02+8.125\times10^{-5}$(AADT-400)	1.15
	6 ft. (base)	1	1	1
	8 ft. or more	0.98	$0.98-6.875\times10^{-5}$(AADT-400)	0.87
Converted CMF	0 ft. (base)	1	1	1
	2 ft.	0.97	$0.96-6.63\times10^{-5}$(AADT-400)	0.87
	4 ft.	0.92	$0.92-9.96\times10^{-5}$(AADT-400)	0.77
	6 ft.	0.91	$0.90-1.5\times10^{-4}$(AADT-400)	0.67
	8 ft. or more	0.89	$0.88-1.93\times10^{-4}$(AADT-400)	0.58

*: HSM Part D (1)

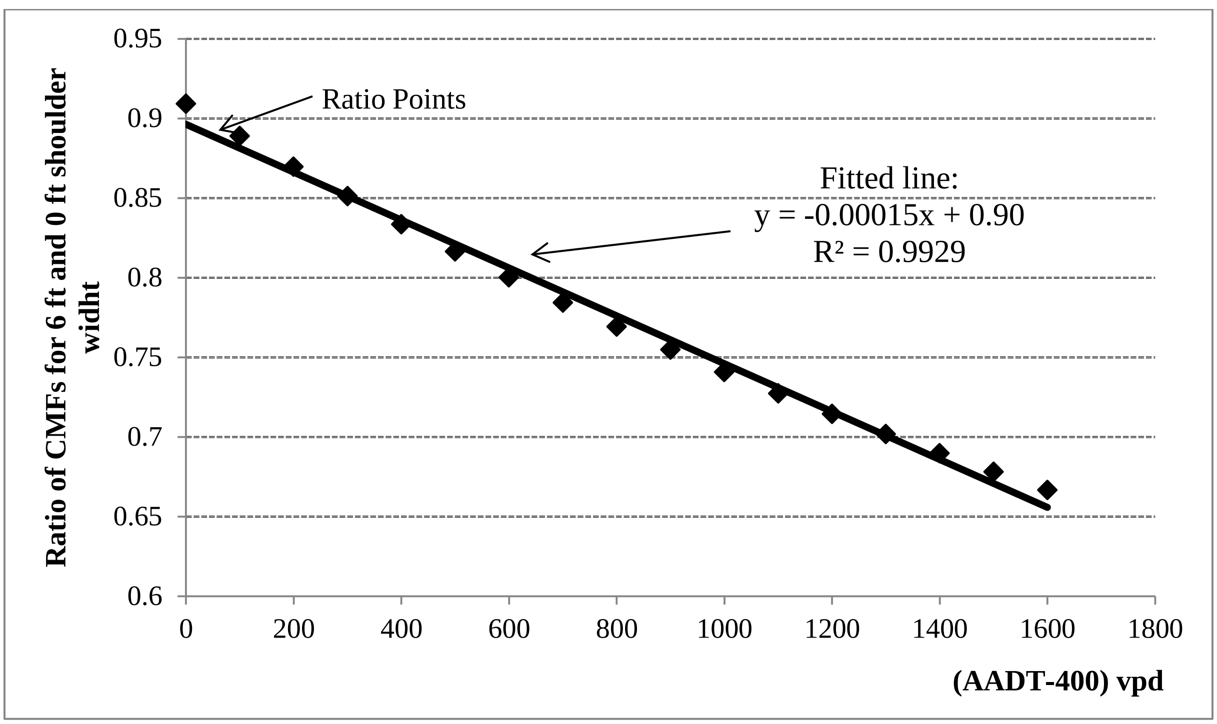

Figure 5.2 Adjusted CMF for 6ft shoulder width

5.1.2.2 Jurisdiction-specific Crash Distribution of CMFs

Converting CMFs to the new base conditions constitutes part of the process. The rest of the calibration depends on the safety effectiveness of individual countermeasures. Not every safety treatment will reduce crashes of all types. For example, widening shoulders may decrease single-vehicle run-off-the-road and opposite-direction crashes, but is not necessarily effective in reducing other types of crashes. Crash-specific CMFs can be changed by replacing default values embedded in HSM predictive models. Most of the default values are proportions of crash severity levels and crash types for estimating the reduction of a specific type of collision (such as nighttime vs. daytime, pedestrian-related crashes vs. others, bicycle crashes vs. others, etc.). Using a jurisdiction-specific crash distribution provides more reliable estimates since a CMF usually contributes to specific crash types.

Table 5.5 shows large disparities between the two distributions. The distribution of collision types, such as single-vehicle run-off-the-road, multiple-vehicle head-on, and sideswipe crashes, affects CMFs for lane width, shoulder width and shoulder type. The proportion of total crashes constituted by relevant crashes p_{ra} was estimated to be 0.574 based on the HSM default distribution, but it was 0.263 based on the local-derived distribution. Hence, the default distribution was replaced with local-derived values throughout the methods.

Table 5.5 Distribution of collision type for rural two-lane two-way roadways

Crash Type	Percentage of Roadway Segment Crashes by Severity					
	HSM*			South Dakota		
Single-vehicle Crash	Fatal+Injury	PDO**	Total	Fatal+Injury	PDO	Total
Collision with Animal	3.8	18.4	12.1	2.0	43.4	33.2
Collision with Bicycle	0.4	0.1	0.2	0.4	0.0	0.1
Collision with Pedestrian	0.7	0.1	0.3	0.9	0.0	0.2
Overturned	3.7	1.5	2.5	7.5	1.4	2.9
Ran off Road	54.5	50.5	52.1	48.4	15.8	23.8
Other Single-vehicle Crashes	0.7	2.9	2.1	31.1	30.5	30.6
Total Single-vehicle Crashes	63.8	73.5	69.3	90.3	91.1	90.9
Multiple-Vehicle Crash						
Angle Collision	10.1	7.2	8.5	3.7	4.5	4.3
Head-on Collision	3.4	0.3	1.6	2.9	0.4	1.0
Rear-end Collision	16.5	12.2	14.2	2.0	2.3	2.2
Sideswipe Collision	3.8	3.8	3.7	1.1	1.6	1.5
Other Multiple-vehicle Crashes	2.6	3	2.7	0.0	0.1	0.0
Total Multiple-Vehicle Crashes	36.2	26.5	30.7	9.7	8.9	9.1
Total Crashes	100	100	100	100	100	100

*: HSM Appendix Table A-3.[1]
**: PDO is short for Property Damage Only.

5.1.3 Calibration Factor C

The final step is a calibration factor C which accounts for all unavailable factors contributing to the disparity between predicted and observed crash frequencies (ex: weather, driver and animal populations, crash reporting threshold and practices). A greater-than-one C means more crashes are observed than expected. A smaller-than-one C means fewer crashes are observed than expected. The calibration factor C for Calibrated HSM, Model 1, 2, 3, and 4 are 1.5368, 0.8058, 0.8565, 0.9998, and 0.9963, respectively.

5.1.4 Method Comparison

After applying SPFs from Model 1 to Model 4 in the IHSDM with adjusted CMFs, calibration factors and predicted crash frequencies were generated for each model. The HSM recommends that at least 30 to 50 sites representing a total of at least 100 crashes per year should be randomly selected. However, there were only 91 crashes in three years, or about 30 crashes per year, which is far below the recommendation. All data was used in the calibration of SPFs. The predicted crash frequency of each site was multiplied by the calibration factor to get the expected crash frequency which was the crash frequency after calibration. The total number of the expected crashes of all sites should be equal to the observed crash counts.

5.1.4.1 Graphical Comparison

Figure 5.3 shows the expected crash frequency after calibration versus AADT for different models. It is assumed that the segment length is one mile and each segment meets the adjusted base conditions. Note that the CMF for lane width was applied to the calibrated HSM method since the base conditions have changed.

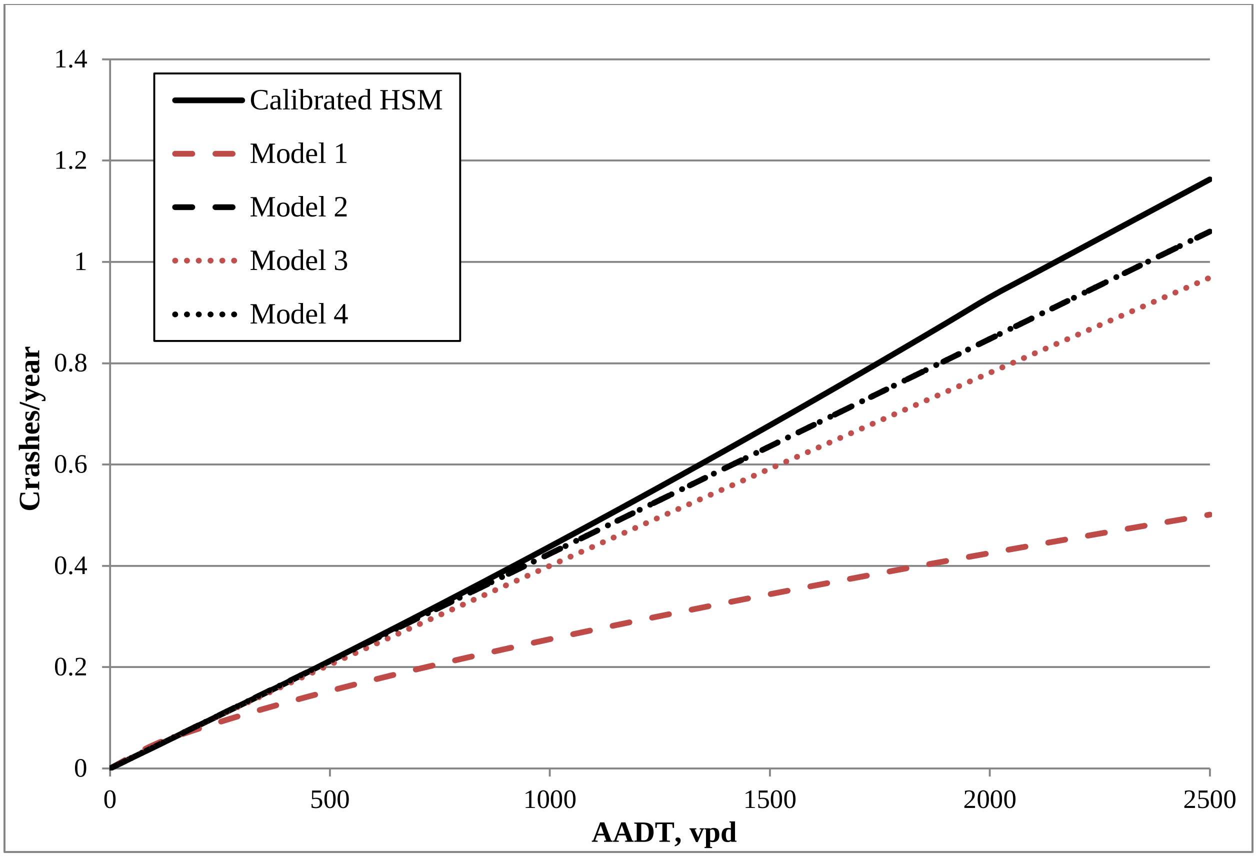

Figure 5.3 Comparisons of different models

When AADT is very low (i.e. less than 400), the difference between models is very small. The difference increases gradually as AADT increases except for Model 1 which has a decreasing rate of change in crash frequency. Both Models 2 and 4 are linear and almost overlap. Model 3 is very close to linear.

Figure 5.4 shows the difference between observed and expected crash frequency against observation for each model. All diagrams are similar in that all models overestimate crash frequency for zero observation sites, all models underestimate for sites with one recorded crash, and all models underestimate crash counts for sites with more than one crash. It is difficult to decipher which model performs better.

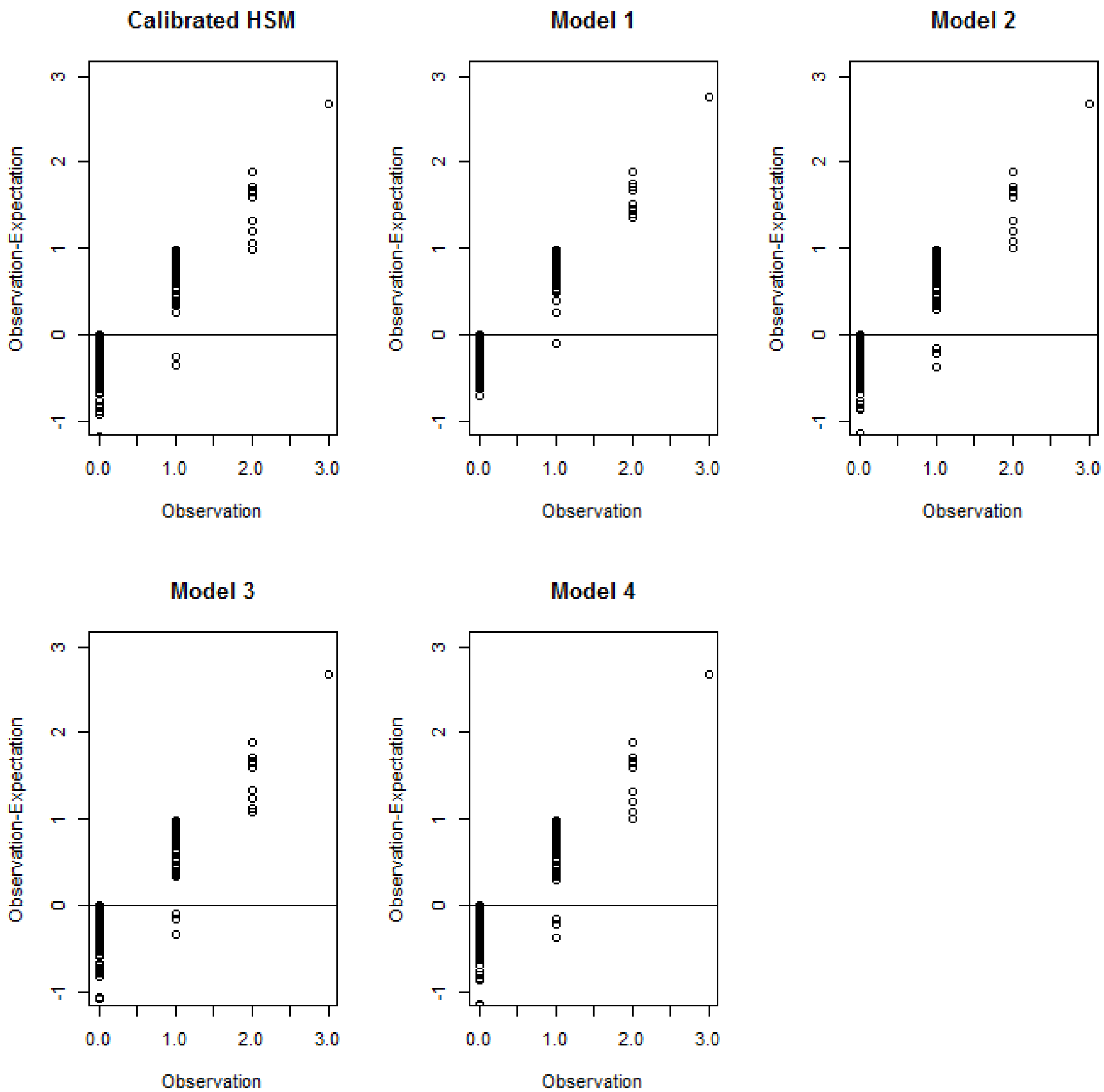

Figure 5.4 (Observation - expectation) vs. observation for each model

5.1.4.2 Statistical Comparison

The following statistical measures were utilized to evaluate the performance of different models. The summary of measured results is presented in Table 5.6.

Table 5.6 Summary of results

	Calibrated HSM	Model 1	Model 2	Model 3	Model 4
Calibration Factor (C)	1.5368	0.8058	0.8565	0.9998	0.9963
Correlation coefficient (ρ)	0.363	0.339	0.365	0.364	0.365
Mean absolute deviation (MAD)	0.2058	0.2153	0.2059	0.2068	0.2058

The calibrated HSM has the largest value of calibration factor amongst all models. This calibration factor is also larger than one, suggesting more crashes occur in South Dakota than predicted by the HSM. Model 1 and Model 2, both of which were developed using data that conforms to new base conditions, have calibration factors that are not as close to one as those of Model 3 and Model 4, both of which were developed with the entire crash data set. In fact, calibration factors from Model 3 and Model 4 are very close to one, implying superior prediction power of crash frequencies. Generally speaking, the correlation coefficient, ρ, between observed and expected crash frequencies for all methods is low. The plausible reason is that for all zero observations which hold a very high proportion in SD rural local two-lane two-high roads, the corresponding predictions are always greater than zero. Model 1 has the largest MAD value and the other four models have very close MAD values, indicating similar prediction performance.

Except for Model 1, all methods have similar correlation and MAD values and varying calibration factors. Since the calibration factor is used to account for the effect of unmeasured or unobserved factors such as crash reporting threshold, driver and animal population, climate, etc., it may be questionable to assume that these factors are similar for all South Dakota sites. The larger the calibration factor, the more the prediction deviates from the observation. Hence, Models 3 and 4 with lower calibration factor are considered to be the better methods.

Among all models, Model 4 not only produces a calibration factor very close to one, but also has the simpler form and better prediction performance. It is therefore considered the best model.

5.2 Modules for Tribal Rural Roads

A total of 56 tribal rural roadway segments were included in the study and, from 2009 to 2011, 41 crashes happened on these segments. Road characteristic information for these segments has been collected and the descriptive statistics of numeric independent variables are shown in Table 5.7.

Table 5.7 Descriptive statistics of data variables

Variable	Mean	Minimum	Maximum	Median	Standard Deviation
Segment Length (mi)	3.563	0.329	17.401	2.387	3.325
AADT (vpd)	678.4	20.7	4123.3	274.7	856.6
Lane Width (ft)	12.9	9	16	12	1.5
Shoulder Width (ft)	0.63	0	8	0	1.60
Speed Limit (mph)	53.4	25	65	55	7.1

Only two out of 56 sites have the identical HSM base conditions defined for rural two-lane two-way roads. The most representative type of road conditions are 12-ft lane widths with no shoulders, and the corresponding number of this type of segments is 15. Since this number is so small, it's not effective to build new SPFs with this sub-dataset. Hence, new SPFs were only developed with the whole dataset.

5.2.1 Safety Performance Functions (SPFs)

The predicted crash frequency generated by IHSDM was 91.1. The calibration factor was computed as 0.450. The calibrated HSM SPF without any modifications is shown in Equation 3.

$$N_{SD\ Tribal} = 0.450 \times AADT \times L \times 365 \times 10^{-6} \times e^{(-0.312)} = 1.202 \times 10^{-4} \times AADT \times L \quad (3)$$

5.2.1.1 Development of Jurisdiction-specific SPFs

Jurisdiction-specific SPFs were also developed with the entire data set. Variables used to develop SPFs include AADT, segment length, lane width, surface type, shoulder width, shoulder type, terrain, and speed limit. Models with poor goodness-of-fit and unreasonable signs for independent variables were rejected and the final model shows only AADT and segment length could produce logical and reasonable results. Two negative binomial models with different forms are shown in Table 5.8.

Table 5.8 Parameter estimate for model 5 and 6

	Model 5 ($N = \beta_0 \times AADT^{\beta_1} \times L$)		Model 6 ($N = \beta_0 \times AADT \times L$)	
Parameter	Estimate	p-Value	Estimate	p-Value
Intercept [$\ln(\beta_0)$]	-5.6916	<.0001	-8.5870	<.0001
Log(AADT) [β_1]	0.5196	0.0011	–	–
Overdispersion	0.64		1.26	
Formula	$N = AADT^{0.5196} \times L \times 3.374 \times 10^{-3}$		$N = AADT \times L \times 1.865 \times 10^{-4}$	

Because the base conditions for new SPFs are the same as those defined in HSM, there is no need to calibrate CMFs. The default HSM crash distribution was substituted with South Dakota's crash distribution shown in Table 5.5 to improve the predictive ability.

5.2.2 Calibration Factor C

The final step is to generate calibration factors for each model. Since the tribal roadway data cannot meet the standard in HSM which recommend that at least 30 to 50 road segments with 100 crashes in total per year should be used to conduct the calibration, all data was used to calibrate SPFs. The calibration factor C for Calibrated HSM, Model 5 and 6 are 0.450, 0.8630, and 0.6447, respectively.

5.2.3 Method Comparison

For each method, SPFs and CMFs were subsequently applied to get predicted crash frequency of each segment. Then crash frequency was multiplied by the calibration factor to generate the expected crash frequency of each site.

5.2.3.1 Graphical comparison

Figure 5.5 shows the expected crash frequency after calibration versus AADT for different models. It is assumed that the segment length is 1 mile and each segment meets the base conditions defined in HSM. Seen from Figure 5.5, the calibrated HSM method and Model 6 are both linear and almost overlap with each other. Model 5 has a decreasing rate of change in crash frequency. When AADT is low (less than about 800 veh/day), Model 5 predicts higher crash frequency than the other two methods. When AADT is greater than about 800 veh/day, Model 5 predicts lower crash frequency than the other two methods and the difference gets larger as AADT increases. When AADT approaches 4,000 veh/day, the calibrated HSM method and Model 6 predict crash frequency more than twice as that predicted by Model 5. This huge difference might be generated by one outlier with an AADT of more than 4,000 veh/day with only one crash during three years while all other sites have AADTs of less than 2,500 veh/day.

Model 5 may provide a more accurate prediction because the relationship between crash frequency and AADT is usually non-linear; an advantage of the calibrated HSM and Model 6 are their simple forms.

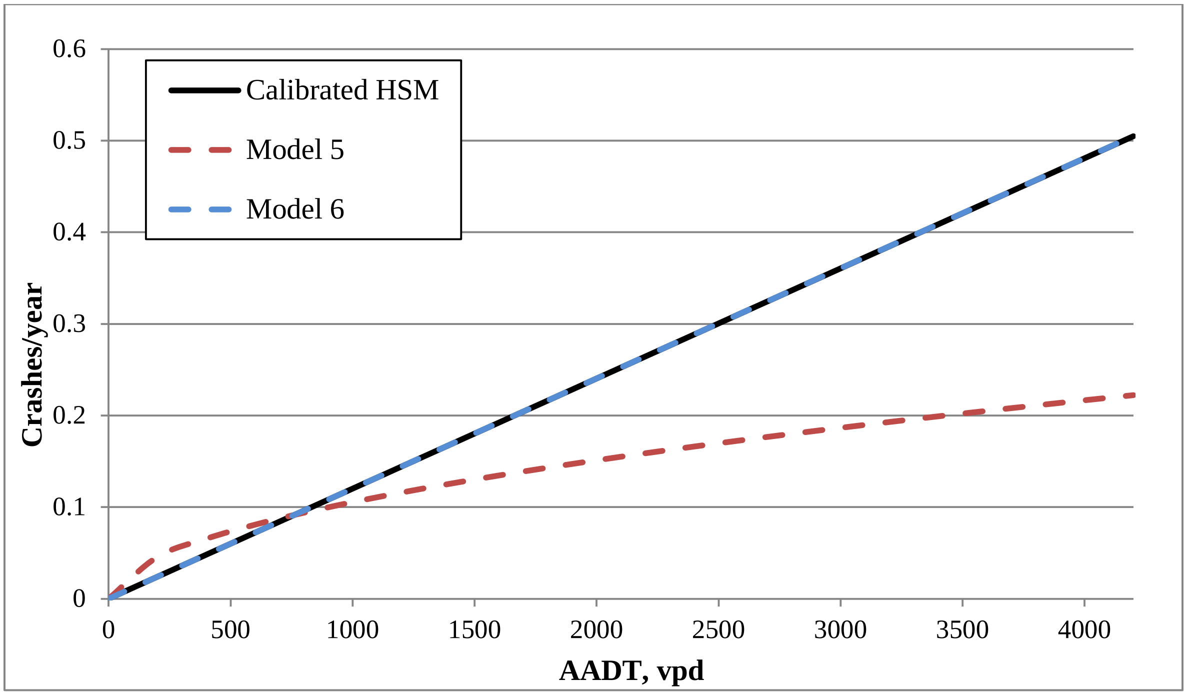

Figure 5.5 Comparisons of different models

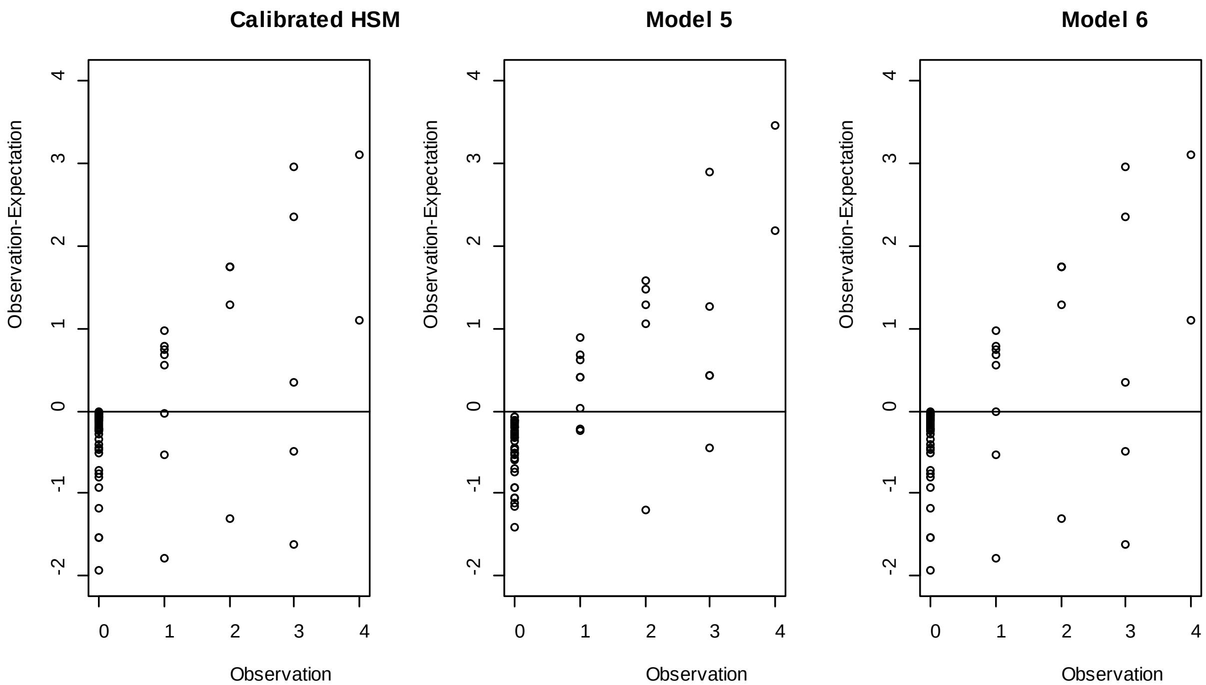

Figure 5.6 (Observation - expectation) vs. observation for each model

Figure 5.6 shows the difference between observed and expected crash frequency against observation for each model. All diagrams are similar in that all models overestimate crash frequency for zero observation sites, all models generally underestimate for sites with one or more than one recorded crash. It is difficult to determine which model performs better.

5.2.3.2 Statistical Comparison

The following statistical measures were utilized to evaluate the performance of different models. The summary of measured results is presented in Table 5.9.

Table 5.9 Summary of results

	Calibrated HSM	Model 5	Model 6
Calibration Factor (C)	0.4500	0.8630	0.6447
Correlation coefficient (ρ)	0.545	0.575	0.545
Mean absolute deviation (MAD)	0.719	0.680	0.719

All calibration factors are smaller than one, suggesting fewer crashes occur on tribal rural roads in South Dakota than predicted by the HSM. Model 5 has the largest value of calibration factor amongst all models. This calibration factor is also closest to one, indicating that Model 5 has the superior predictive ability. The correlation coefficient, ρ, between observed and expected crash frequencies for all methods is generally not high and Model 5 has the largest correlation coefficient. The possible reason is that for all zero observations which hold a very high proportion, all models always predict crash frequencies higher than zero. Model 5 has the smallest MAD value and the difference of all MAD values is small, indicating similar prediction performance.

All methods have similar correlation and MAD values and varying calibration factors. Because the calibration factor is used to account for the effect of unmeasured or unobserved, it may be questionable to assume that these factors are similar for all tribal rural sites. The further away from one the calibration factor is, the more the prediction deviates from the observation. Hence, Model 5 is considered to be the best method.

6. PROJECT EVALUATION

After developing locally derived safety modules, design project alternatives can be evaluated with these modules by comparing their safety performance.

To illustrate the evaluation procedure, the same 4-mile long roadway section of USH 83 used in Section 6 was utilized. The current conditions of this roadway are that it has 12-ft lanes, 1-ft paved shoulders, no centerline rumble strips, and no lighting. The speed limit on this roadway is 35 mph, and AADT is 305 veh/day.

This roadway is not located in tribal area, so it's treated as a rural local roadway. And locally specific safety modules can be used to evaluate this roadway. Crash frequency from 2013 to 2015 is considered as the safety performance. With current conditions, the modules predict 3.98 crashes in total along the roadway during three years. Figure 6.1 shows the crash rate by segment. Several spikes in the figure show corresponding segments which experienced high crash rates greater than 1 crash per mile per year. These segments are identified as high-risk segments.

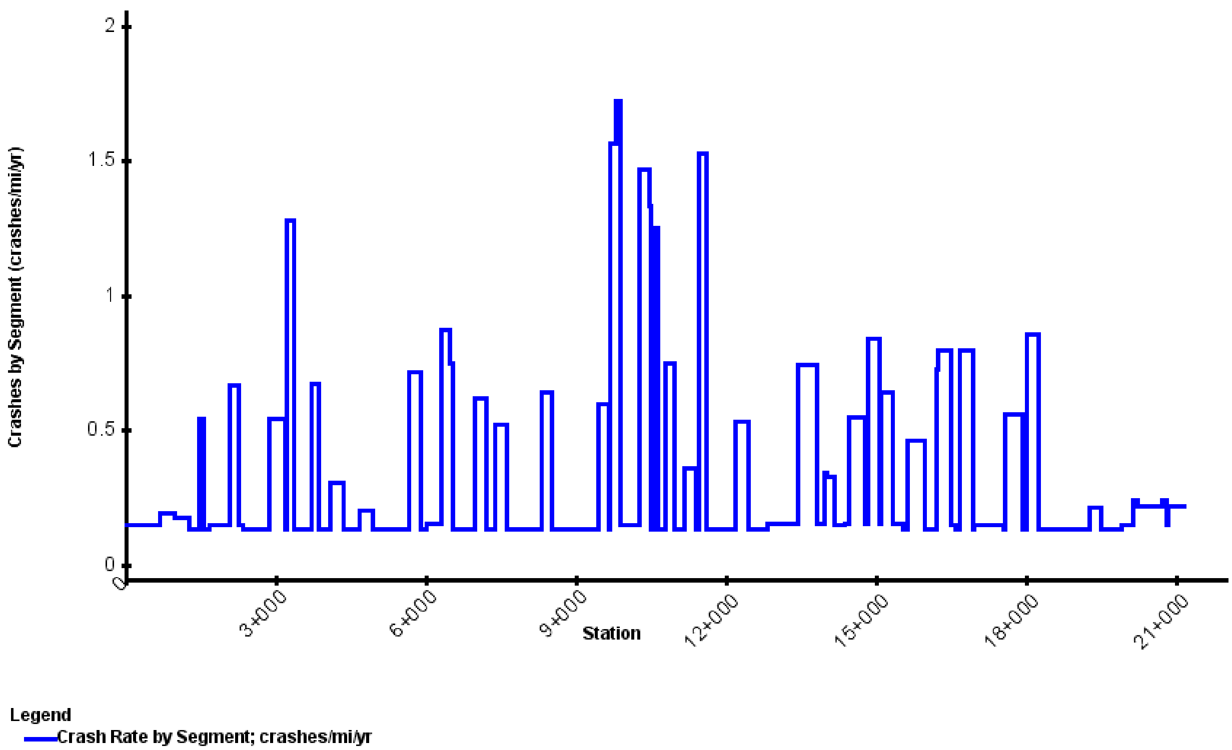

Figure 6.1 Crash rate by segments with current conditions

Two alternatives were designed on this roadway. One is widening shoulder width all along the roadway; the other is adding one passing lane at high-risk segments.

6.1 Alternative 1

Table 5.5 shows that single-vehicle run-off-the-road, and multiple-vehicle head-on, and sideswipe crashes make up 26.3% of all crashes, and widening shoulder width is an effective way to reduce the three types of crashes. Currently the shoulder width is 1 ft., and the alternative is to widen it to 4 ft. After widening the shoulder width to 4 ft. all along the roadway, the crash count predicted by the modules is 3.84. Due to the low volume (<400vph) on the road, CMF for widening shoulder width from 0 ft. to 4 ft. is 0.92 according to Table 5.4. Considering only 26.3% of all crashes that may be reduced, the crash frequency is predicted to be reduced by only 3.5%. Figure 6.2 shows the crash rate after widening the shoulder width. The trend in Figure 6.2 looks the same as that in Figure 6.1.

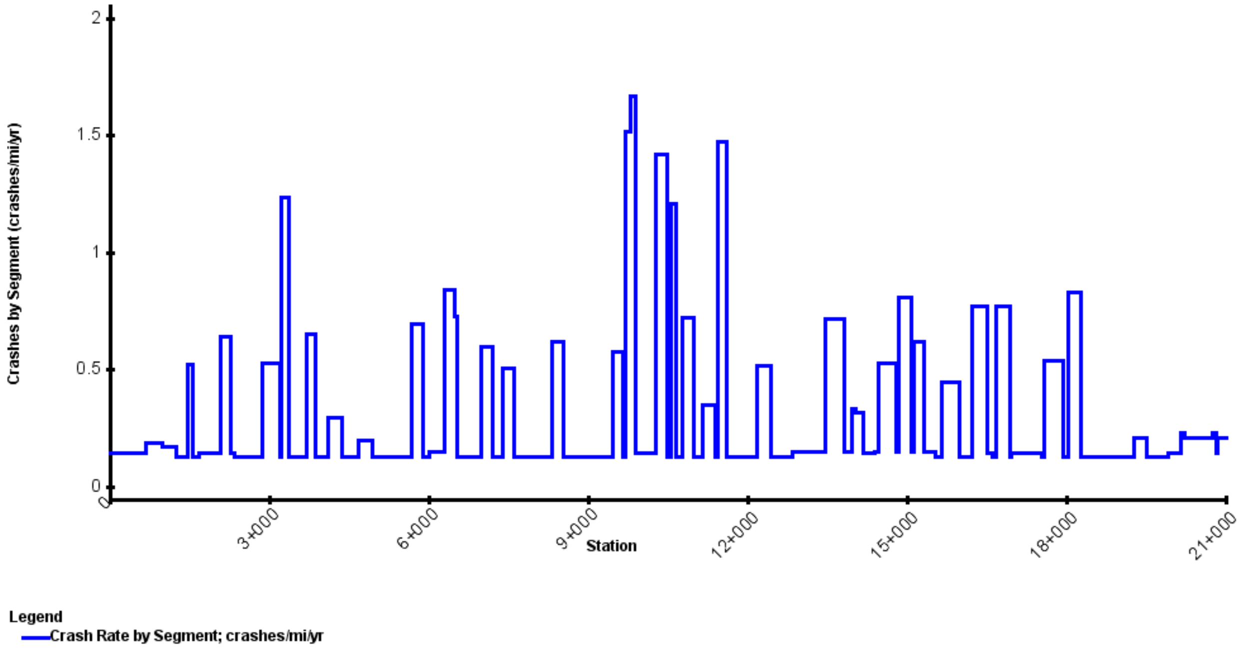

Figure 6.2 Crash rate by segments after widening shoulder width

6.2 Alternative 2

In this alternative, one passing lane is added to help improve the safety condition of high-risk segments. Table 6.1 gives the information of all high-risk segments.

Table 6.1 Stations and crash information of high-risk segments

Start Location	End Location	Length (mi)	Expected Number of Crashes	Crash Rate (crashes/mi/yr)
3+220.800	3+326.400	0.02	0.077	1.2807
3+326.400	3+370.800	0.0084	0.032	1.2807
9+694.080	9+794.400	0.019	0.089	1.5673
9+794.400	9+884.705	0.0171	0.088	1.7241
10+264.320	10+486.080	0.042	0.185	1.4686
10+486.080	10+493.070	0.0013	0.005	1.3351
10+554.720	10+661.863	0.0203	0.076	1.2527
11+452.320	11+605.853	0.0291	0.133	1.5255

The table shows that other than two short high-risk segments located at about 3,000 ft. from the beginning of the road all other high-risk segments are located from 9,000 ft. to 12,000 ft. According to the AASHTO design guidebook,Error! Reference source not found. the optimal length of passing lanes is 0.5-2.0 mi. To effectively reduce the crashes on high-risk segments and efficiently spend the funding, the passing lane is added to cover those high-risk segments from station 9,000 ft. to station 12,000 ft. and the final decision is made to add one passing lane from station 9+461.760 ft. to station 12+451.377 ft., for 0.5662 mi in total. This road section has 0.37 crashes/mi/yr on average.

After adding one passing lane to one portion of the road, the predicted crash frequency becomes 3.74 and is reduced by 6.0% compared with original crash frequency.

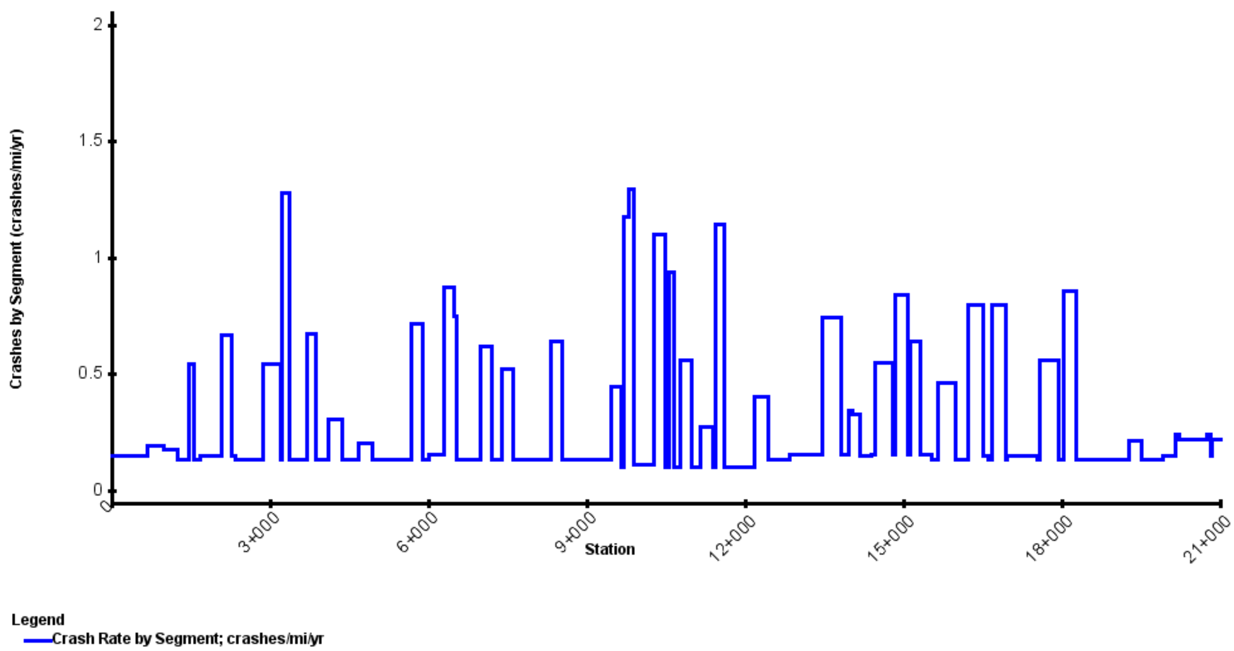

Figure 6.3 Crash rate by segment after adding passing lanes

Figure 6.3 shows the crash rate at each segment after adding passing lanes. It can be seen that the general trend becomes smoother because the crash frequencies at the section with intense high-risk segments are greatly reduced.

6.3 Evaluation

The two alternatives target approach crash reduction from two different aspects: reducing the general level of crashes and reducing crashes at high-risk segments. Improving the conditions all along the roadway may cost more money but can improve the overall safety; improving safety at high-risk segments is more cost-effective, but may not make other segments safer. To effectively evaluate these two alternatives, the detailed cost should be assessed and then a benefit-cost analysis should be conducted.

7. CONCLUSION

The purpose of this study was to develop locally derived IHSDM safety modules for South Dakota and North Dakota. To achieve this goal, the availability of data for rural local roads and tribal rural roads were evaluated and obstacles to implementing the modules were resolved. After the modules were developed, design alternatives were evaluated based on the safety performance using the modules.

Data needs assessment was conducted before requesting the appropriate data. Safety data were obtained from the SDDOT and NDDOT. Road data and crash data are both in GIS format, but one appears as line features and the other as point features. To develop modules, two types of data need to be combined by using spatial tools in GIS. Lots of manual work was also required to clean the errors in the combined data to obtain accurate results. So far, the curve data has not been available, and methods were proposed to collect it such as collecting data through GPS, GIS and imagery files. The level of precision and the level of effort needed were evaluated for each method. Despite the promising prospect of collecting curve data from data sources other than field survey, the semi-manual data collection at a large scale is still a formidable task and therefore, was not carried out in this project. These methods may become more efficient via computer programming.

HSM was studied thoroughly to gain better understanding of the knowledge for developing jurisdiction-specific SPFs, modifying CMFs and calibrating SPFs. Locally derived modules were developed for both rural local roads and tribal rural roads. Modules for rural local roads were developed using 657 segments in South Dakota. The calibration factor using HSM SPF is 1.537, indicating that more crashes occur on South Dakota's rural two-lane two-way roads than predicted by HSM. Base conditions for SPF were modified to better represent the situations in South Dakota. With two different datasets (the whole dataset and the sub-dataset conforming to new base conditions and two different formula forms) four different jurisdiction-specific SPFs were built. CMFs were adjusted to new base conditions, and crash distribution in South Dakota was derived from local data to advance the predictive power of the modules. Finally, five methods, including calibrated HSM method and four local-derived methods, were compared and the best method was the model with the same form of SPF in HSM derived from the whole dataset. The best method not only generates a calibration factor very close to one, but also has good statistical measurements.

Modules for tribal rural roads were based on 56 tribal rural roads in South Dakota. The generated calibration factor for HSM SPF is 0.450, which implies that fewer crashes occur on South Dakota's tribal rural roads than predicted by HSM. The size of the most representative type of segments was too small to develop meaningful SPFs. Therefore, new SPFs were built based on the whole dataset. Two negative binomial models and the calibrated HSM method were compared, and the new model using an exponential from of AADT was determined as the best method.

Using the locally-derived modules, two design alternatives for one 4-mile long portion of USH 38 highway were evaluated based on the safety performance, illustrating the capability of developed modules for project evaluation.

This study provides guidance and empirical results on how to calibrate IHSDM models for local agencies. The calibration processes and procedures will be expanded to other highway facilities. Unavailable data, such as curve and driveway density, should be collected to develop more accurate and reliable jurisdiction-specific SPFs. Moreover, separate calibration factors may be considered for regions with distinct features such as mountain vs. plain, dry vs. wet, or as a function of AADT or other characteristics.

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