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SAFETY PERFORMANCE FUNCTIONS FOR FREEWAY MERGE ZONES

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16. Abstract
This report documents the results of a research project to support CDOT in the area of Safety Performance Function (SPF) development. The project involved collecting data and developing SPFs for ramp-freeway merge zones categorized as isolated, non-isolated and weave. For each of these three categories, data for the period 2007 to 2011 were collected at sites selected to ensure statewide geographical representation and coverage of the range of traffic volume and other variables in each category. The development of SPFs for the three categories of ramp-freeway merge zones was successful. Separate SPFs were developed for Total, fatal+injury (FI) and Property Damage Only (PDO) crashes.

Implementation
CDOT can use the developed SPFs immediately to apply state-of-the-art methodologies for road safety management activities.

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EXECUTIVE SUMMARY

The Colorado Department of Transportation's (CDOT's) research and safety engineers are in the forefront of national efforts to develop methods that use Safety Performance Functions (SPFs) to screen large networks to find sites with a potential for safety improvement. CDOT has previously developed SPFs to identify freeway, rural roadway segments, ramp terminals, and ten categories of intersections that have the potential for crash reduction. This report documents a further effort to support CDOT in the area of SPF development.

This effort involved the data collection and development of SPFs for three categories of ramp-freeway merge zones, classified as isolated, non-isolated and weave. For each category, data were collected at sites selected to ensure statewide geographical representation and coverage of the range of traffic volume and other variables in each category. Data were collected for the period 2007 to 2011.

The development of SPFs for the three categories of ramp-freeway merge zones was successful. Separate SPFs were developed for Total, fatal+injury (FI) and Property Damage Only (PDO) crashes. The SPFs contain logical variables with intuitive directional effects.

It was not feasible to collect data for all locations of interest under CDOT's jurisdiction due to budget constraints. The sites pursued for this project were limited to those where a ramp average annual daily traffic (AADT) was already available. It is recommended that data for additional sites be collected as they may become available. Such data can be used to screen the entire network and ultimately to enhance the developed SPFs. As more years of crash and traffic data become available, these data can be added to the database to continually update information. The SPFs can be recalibrated to apply to these additional years of data using a procedure documented in this report. When several additional years of data and sites are available, it may be desirable to calibrate a new set of original SPFs.

Implementation Statement

CDOT can immediately use the developed SPFs to apply state-of-the-art methodologies for road safety management activities. CDOT can apply the SPFs developed to screen network applications, diagnose crash problems at specific sites and conduct before-after evaluations of implemented treatments. CDOT can also incorporate the SPF mathematical equations into software and other tools and methodologies that CDOT currently uses to screen the road network for sites with the greatest potential for safety improvement, to develop treatments and to evaluate the effect of improvements using state-of-the-art methods.

If crash predictions are desired for two or more severities out of Total, fatal+injury (FI) and PDO, it is recommended to use the SPFs for PDO and FI crashes directly. If one of the desired severities is Total, then add the estimates for PDO and FI. The reason is that at some rare extremes of the variables included in the SPFs the estimate for PDO or FI could be greater than Total.

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1.0 INTRODUCTION

Road safety management activities include screening the network for sites with a potential for safety improvement (Network Screening), diagnosing safety problems at specific sites and evaluating the safety effectiveness of implemented countermeasures. It is important that these activities be both efficient and methodologically sound because resources would otherwise be wasted on unnecessary treatments for safe elements and elements deserving of treatment would be left untreated.

The state-of-the-art methodologies for conducting these activities use statistical models to predict expected crash frequencies using traffic volumes and other site characteristics as the input to the models (known as Safety Performance Functions or SPFs). The following is an example of an SPF for a road segment:

$$\text{Crashes/mile/year} = (\text{alpha})\cdot(\text{AADT})^{b1}$$

Where,

- alpha and b1 are parameters estimated in the modeling process;
- AADT is the estimated average annual daily traffic volumes on the roadway

CDOT’s research and safety engineers are in the forefront of national efforts to develop methods using SPFs to screen large networks to find sites with a potential for safety improvement. CDOT has previously developed SPFs to identify freeway, rural roadway segments, 10 categories of intersections and ramp terminals at diamond interchanges that have the potential for crash reduction. There still remain, however, other site types for which no SPFs are available. This report documents an effort to develop SPFs for merge zones for freeway on-ramps. The mitigation of crashes at on-ramp lanes can be accomplished by safety treatments such as ramp metering or design modifications. Thus, it is desirable to develop SPFs for these types of facilities.

This report documents the data collection, modeling efforts and findings of a research project to develop SPFs for on-ramp merge zones. It was not feasible to collect data for all such locations under CDOT’s jurisdiction due to budget constraints. The sites pursued for this project were limited to those where a ramp AADT was available.

2.0 DATA ASSEMBLY

The data collection phase of the project involved developing a database of on-ramp merge areas suitable for developing SPFs. Data collection involved collecting geometry, traffic volume and crash data on both the on-ramp and mainline. The following tasks were undertaken:

1. Compiled a list of existing ramps, including location, increasing or decreasing direction and ramp speed limit.
2. Compiled existing on-ramp AADT data from 2011.
3. Queried freeway mainline data for mainline AADTs adjacent to the on-ramps from 2002 to 2011, as well as the peak percent truck traffic, the design hour volume factor, the directional distribution factor and the mainline speed limit.
4. Using satellite photography added geometric information, including:
    a. Interchange type (for example, loop, diamond, etc.)
    b. Tapered vs parallel merge lane
    c. Ramp type (for example, diamond, parclo loop, etc.)
    d. Urban vs rural location
    e. Length of merge lane from gore to end of taper
    f. Number of lanes on ramp
    g. Number of mainline through lanes upstream of ramp
    h. Number of mainline through lanes downstream of ramp
    i. Presence of on-ramp upstream within 1500 ft.
    j. Presence of off-ramp upstream within 1500 ft.
    k. Presence of on-ramp downstream within 1500 ft.
    l. Presence of off-ramp downstream within 1500 ft.
    m. Presence of a weave area upstream
    n. Presence of a weave area downstream
    o. Length of weave area if present
5. CDOT provided a list of on-ramps operating under a metered control and the date at which this became active.
6. Queried the crash database for all crashes within 1 mile of each on-ramp between the years 2002 and 2011.

CDOT's Safety Engineering and Analysis group provided the crash data. This group maintains a comprehensive set of databases containing detailed crash history and geometric data.

In consultation with CDOT staff, it was decided to limit the study period to the data from 2007 to 2011. Using years of data prior may be biased because CDOT's crash reporting process changed prior to this time. An additional concern is that it is more difficult to ensure there were no other significant changes at a location the further back in time the study period includes. For three on-ramps with a metered operation, the ramp metering became active within the study period. For these three sites, only data after the metering became active were included.

The available AADT data are for both directions of travel. After discussion with CDOT, it was assumed that the AADT is evenly split between both directions of travel. The ratio of the 2011 mainline AADT to the 2007 to 2011 average was used to extrapolate the available 2011 on-ramp AADTs to an average for the 2007 to 2011 period.

The crash data were matched to each on-ramp using the highway number, milepost and direction of travel (primary or secondary). The crashes include those occurring on the mainline as well as the on-ramp within the merge lane. A limited number of crashes had an unknown direction of travel and were not included in the data. These included only approximately 1.5 percent of Total crashes.

Prior to modeling the safety of merge zones, the first step was to define what area constitutes the merge zone. It seems logical that the merging vehicles will have an impact on safety upstream and downstream of the actual merge lane. However, what the length of the merge zone should be was unknown. **Figure 1** illustrates the merge zone area. This area includes the on-ramp in the vicinity of the mainline and some distance upstream and downstream where it could be expected that the merging traffic has an impact on safety.

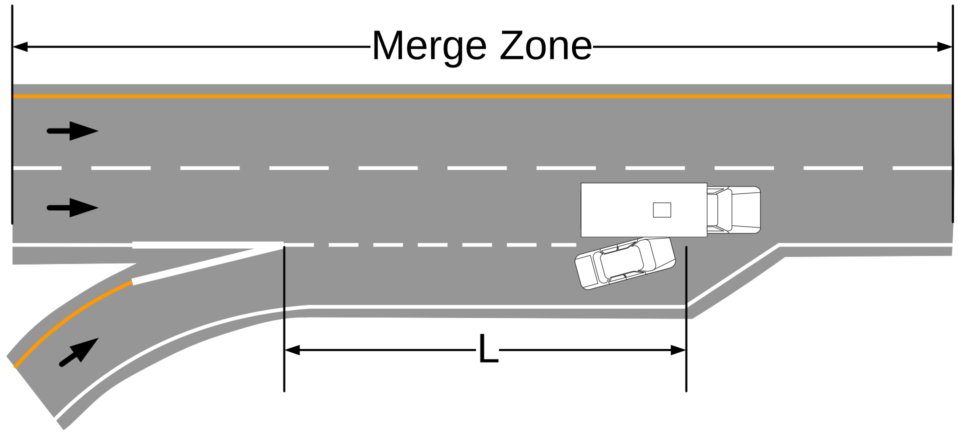

Figure 1. **Merge Zone Diagram**

The situation in **Figure 1** is the most simple and straightforward situation for modeling. However, there were sites with more complicated situations.

Situation A – Locations With a Close On-Ramp or Off-Ramp

As shown in **Figure 1**, the merge zone is thought to extend some distance upstream and downstream of the merge lane. However, if the merge zone extends far enough, it then encompasses the upstream/downstream off-ramp and potentially an upstream/downstream on-ramp depending on the interchange configuration. In these cases, the influence areas of each ramp are overlapping.

For these locations it is not possible to define a merge zone that is not affected by and affecting the adjacent areas. Assuming a mainline travel speed of 65 mph (95 ft./s), 1500 ft. is only approximately 16 seconds of travel time.

Situation B – Locations That Are Lane Additions/Weaves

Ninety-two locations are not merge zones as illustrated in **Figure 1**. These locations are weave areas where the on-ramp extends through to a downstream off-ramp. In some cases, the distance between the on-ramp and downstream off-ramp is quite large and the section is really functioning as a lane addition/lane drop. Seventy-four of these sites with a gore to gore distance of up to 2500 ft. could be classified as weave sections in accordance with the Highway Capacity Manual (HCM) definition.

To investigate an appropriate merge zone definition, alternate influence areas were evaluated by adding a distance, x, upstream of the merge lane gore and also downstream of the end of the merge lane taper. Within each influence area, the crash rate per mile-yr. was calculated for each site and the average over all sites determined. **Figure 2** graphs the results from a range of x = 0 to 1 mile. Double counting of crashes was eliminated in cases where the influence areas overlapped.

The average crash rate is relatively stable between approximately x = 0.0 to 0.1 miles (528 feet). Thereafter the average crash rate increases until approximately x = 0.3 miles (1584 feet) after which the average crash rate falls. This indicates that many crashes influenced by the merge lane do in fact occur outside the strict limits of the merge lane and then there is a distance after which the average crash rate decreases, which is intuitive because crash rates are expected to be lower outside the interchange area.

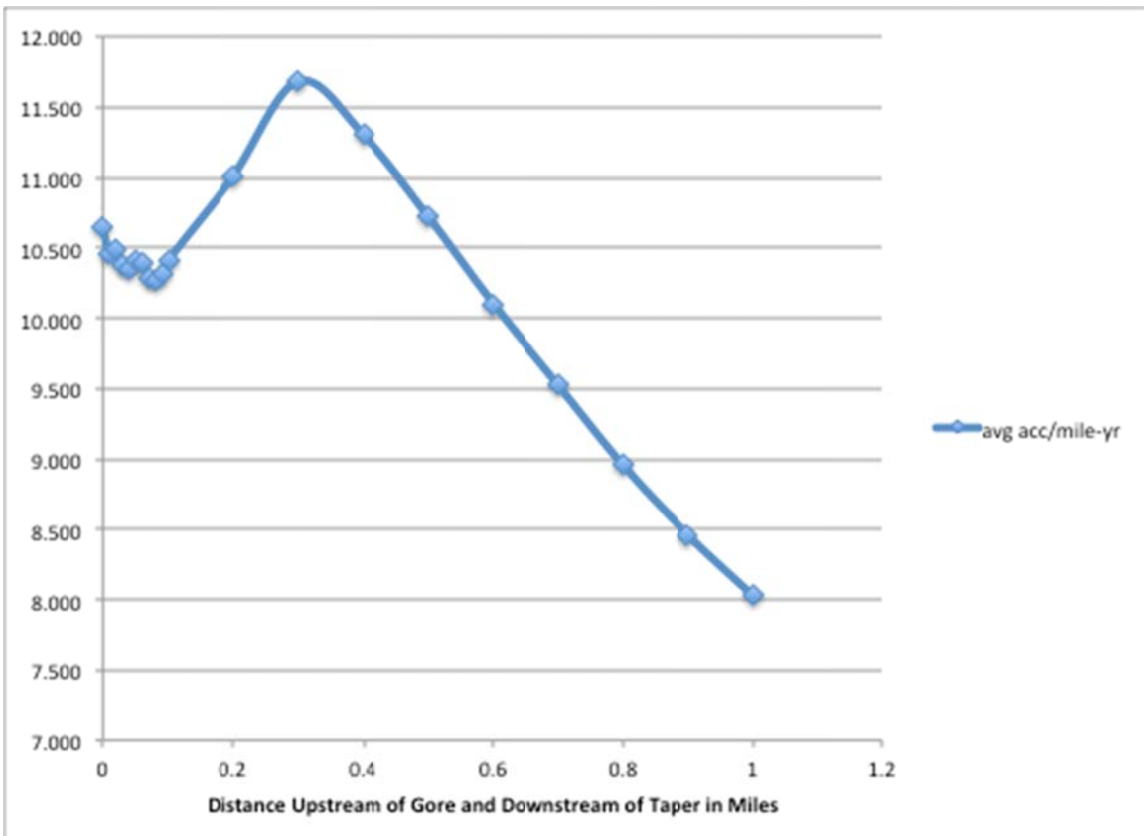

Figure 2. **Average Crash Rates by Influence Area**

Based on the analysis of alternate merge zone definitions, the following decisions were suggested to and approved by CDOT:

1. It was decided to separately model the locations with downstream weaves and those without. These are clearly different situations with different demands on drivers.

2. For those sites without another ramp within 1500 ft., a distance of 1500 ft. upstream of the merge lane gore extending to 1500 ft. downstream of the end of the merge lane taper is used to define the influence area. These are classified as isolated ramp merge zones for SPF calibration purposes.

3. For those sites with another ramp within 1500 ft., where there is a merge lane taper, the distance between the merge lane gore and taper is used to define the influence area exclusively. These are classified as non-isolated ramp merge zones for SPF calibration purposes.

4. For sites where the merge lane extends to a downstream ramp, the gore to gore distance between the two ramps is used to define the influence area. Sections where this distance was up to 2500 ft. were classified as weaving sections for SPF calibration purposes.

In addition to Total crashes, the data were queried to provide fatal+injury (FI) crashes and property damage only (PDO) crashes separately. Separate SPFs were also investigated for late-night crashes given that these crashes may have little to do with traffic volumes and that SPFs for these crashes may show different predictor variables are relevant. Total, FI, and PDO crashes for late-night were also queried, with late-night defined as occurring between the hours of 11 p.m. and 5 a.m.

Tables A and B provide summary statistics for the sites used in developing the SPFs.

Table A. Summary Statistics of Geometric and Traffic Data

Full Description	Mainline AADT			Ramp AADT			Years of Data		
	min	max	mean	min	max	mean	min	max	mean
Weaves (74 sites)	5,640	111,400	51,484	76	14,405	6,120	3	5	4.97
On-ramps with another ramp within 1500 ft. (69 sites)	2,100	124,500	28,709	9	15,891	2,953	3	5	4.97
On-ramps with no other ramps within 1500 ft. (385 sites)	465	110,600	20,424	17	23,436	2,948	2	5	4.99
	Peak Percent Trucks			**Mainline Speed Limit**			**Ramp Speed Limit**		
	min	**max**	**mean**	**min**	**max**	**mean**	**min**	**max**	**mean**
Weaves (74 sites)	2.3	16.2	5.4	45	75	59	45	45	45
On-ramps with another ramp within 1500 ft. (69 sites)	1.8	18.4	8.3	45	75	65	45	45	45
On-ramps with no other ramps within 1500 ft. (385 sites)	1.1	25.2	10.2	45	75	69	40	45	45
	Merge Zone Length/Weave Length (ft.)			**Upstream Lanes**			**Downstream Lanes**		
	min	**max**	**mean**	**min**	**max**	**mean**	**min**	**max**	**mean**
Weaves (74 sites)	243	2503	1,335	2	5	2.96	2	5	2.96
On-ramps with another ramp within 1500 ft. (69 sites)	98	2,684	826	2	6	2.4	2	6	2.4
On-ramps with no other ramps within 1500 ft. (385 sites)	10	3,053	869	1	5	2.2	1	6	2.2
	Lanes on Ramp			**Ramp Type**	**Tapered vs Parallel**	**Urban vs Rural**	**Metered**		
	min	**max**	**mean**	**frequency**	**frequency**	**frequency**	**frequency**		
Weaves (74 sites)	1	2	1.03	Diamond - 63 Parclo loop - 11	Parallel – 74 Tapered - 0	Urban – 64 Rural - 10	Yes – 12 No - 62		
On-ramps with another ramp within 1500 ft. (69 sites)	1	2	1.0	Diamond - 44 Parclo loop - 25	Parallel – 62 Tapered - 7	Urban – 39 Rural - 30	Yes – 3 No - 66		
On-ramps with no other ramps within 1500 ft. (385 sites)	1	3	1.0	Diamond - 382 Parclo loop - 3	Parallel – 289 Tapered - 96	Urban – 133 Rural - 252	Yes – 22 No - 363		

Table B. Summary Statistics of Crash Data

Full Description	Total Crashes/Mile-Year			FI Crashes/Mile-Year			PDO Crashes/Mile-Year		
	min	max	mean	min	max	mean	min	max	mean
Weaves (74 sites)	0.00	568.11	44.20	0.00	61.52	4.68	0.00	509.59	39.23
On-ramps with another ramp within 1500 ft. (69 sites)	0.00	488.48	39.58	0.00	27.08	3.76	0.00	461.40	35.50
On-ramps with no other ramps within 1500 ft. (385 sites)	0.00	72.57	6.51	0.00	5.37	0.62	0.00	69.13	5.88
	Late-Night Total Crashes/Mile-Year			Late-Night FI Crashes/Mile-Year			Late-Night PDO Crashes/Mile-Year		
	min	max	mean	min	max	mean	min	max	mean
Weaves (74 sites)	0.00	19.69	1.77	0.00	3.33	0.32	0.00	17.23	1.45
On-ramps with another ramp within 1500 ft. (69 sites)	0.00	10.91	1.41	0.00	3.64	0.22	0.00	8.00	1.19
On-ramps with no other ramps within 1500 ft. (385 sites)	0.00	6.51	0.80	0.00	1.29	0.12	0.00	5.33	0.68

3.0 STUDY METHODOLOGY

Consistent with state-of-the-art methods, generalized linear modeling, with the specification of a negative binomial (NB) error structure, was used to develop the SPFs. In turn, the specification of an NB error structure allows for the direct estimation of the overdispersion parameter since this is a parameter of the NB distribution. This parameter is used in the empirical Bayes procedure for estimating the expected safety performance of an intersection for various safety management purposes.

SPFs were developed separately for Total, fatal+injury (FI) and Property Damage Only (PDO) crashes. Models for many logical variable combinations were attempted using conventional model forms for exposure and geometric variables. Alternative models were assessed and compared by examining the value of the overdispersion parameter (a smaller value indicates that the model explains more of the variation in collision frequency), the statistical significance of the coefficients, the logical relationship indicated by the coefficients, and in some cases, cumulative residual (CURE) plots. In some cases for fatal+injury crash SPFs, it was decided to retain a variable with low statistical significance if the indicated effect for fatal+injury crashes was in the same direction and of the same order of magnitude as for PDO and Total crashes and if the variable was statistically significant for those crash severities.

In the Cumulative Residuals (CURE) method, documented by Hauer & Bamfo[1], the cumulative residuals (the difference between the observed and predicted values for each site) are plotted in increasing order for each covariate separately. Also plotted are graphs of the 95 percent confidence limits. If there is no bias in the model, the plot of cumulative residuals should oscillate around the x-axis without systematic over or under-prediction, and stay inside these confidence limits. The graph shows how well the model fits the data with respect to each individual covariate. **Figure 3** illustrates a CURE plot for one model for the major road AADT covariate. The indication is that the fit is very good for this covariate in that the cumulative residuals oscillate around the value of zero and lie between the two standard deviation boundaries.

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[1] Hauer, E. and J. Bamfo, "Two Tools for Finding What Function Links the Dependent Variable to the Explanatory Variables." Available at www.roadsafetyresearch.com.

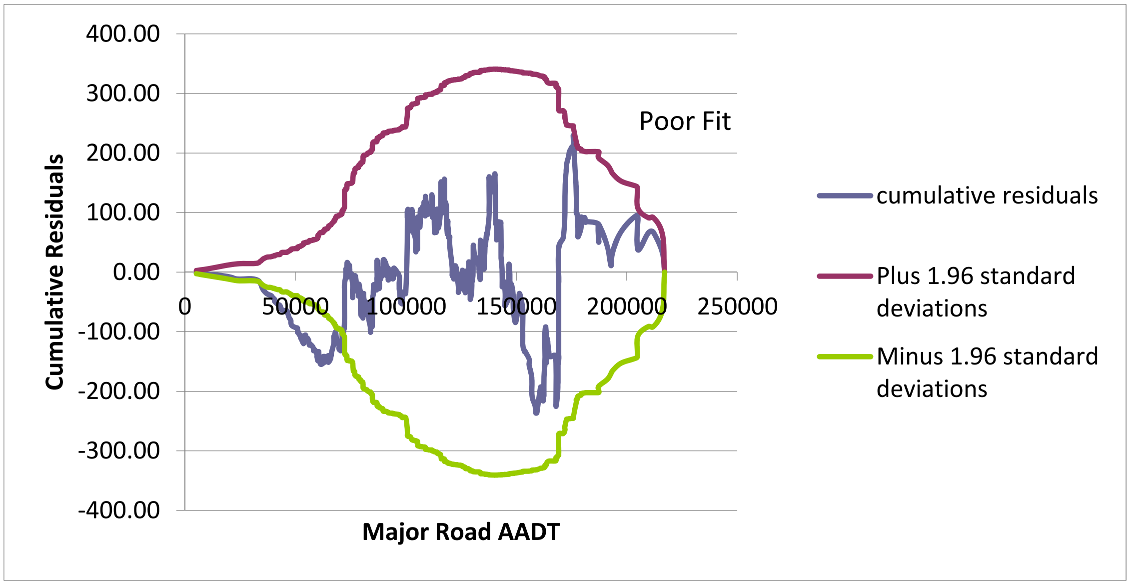

Figure 3. Example of CURE Plot

4.0 SAFETY PERFORMANCE FUNCTIONS CALIBRATED

This section presents the final recommended SPFs developed. Separate SPFs were developed for isolated ramps, non-isolated ramps and weave areas. Separate SPFs were developed for Total, fatal+injury (FI) and PDO crashes.

All of the models have AADT downstream of the ramp as the main exposure variable. Consistent with the operational analysis in the HCM, estimated Lanes 1 and 2 volumes (adjacent to the ramp) were also considered instead of the entire directional mainline AADT, but that investigation, which is documented in Section 4.4, revealed that the entire mainline AADT was a better predictor of crashes in the ramp influence area as defined for this project.

AADT on the ramp was also considered as an additional exposure variable since, logically, expected crash frequency should increase with an increasing ramp AADT. In the modeling, however, the ramp AADT variable was never statistically significant, the influence on expected crash frequency was always small and in some cases actually indicated fewer expected crashes with increasing ramp volumes. It was confirmed by CDOT that the mainline AADT estimates would already include the ramp volumes, which may be the cause of this result. However, even when the ramp volumes are excluded and modeled as a separate exposure term, this variable was found to have a negligible and insignificant effect on expected crash frequency.

4.1 Recommended SPFs for Isolated Ramp Merge Zones

The final model form for isolated ramp merge zones is as follows:

Crashes/year = (merge zone length) $\exp^{(\text{intercept+adjustment term})}\text{AADT}^{b}$

where,

the merge zone length is in miles measured from 1,500 ft. upstream of gore to 1,500 ft. downstream of the end of the merge lane taper

AADT is the mainline AADT count

In **Table C,** “Intercept” and b are the estimated model parameters. Note that the value of the intercept term depends on the type of acceleration lane (parallel or tapered) and on whether there are two or more than two upstream lanes. When the intercept adjustment is applied, its value is simply added to the “Intercept” estimate. If the base case is met the intercept term is not applied.

Table C. Estimated Parameters for Recommended SPFs for Isolated Ramp Merge Zones

Parameter	TOTAL		INJURY		PDO	
	Estimate	Standard Error	Estimate	Standard Error	Estimate	Standard Error
Intercept	-1.8371	0.7292	-3.8104	0.8283	-1.9814	0.7520
b	0.4250	0.0670	0.3676	0.0764	0.4303	0.0691
Intercept adjustment for parallel acceleration lane[1]	-0.2189	0.1271			-0.2283	0.1313
Intercept adjustment for two upstream lanes[2]	-0.3844	0.1722	-0.3161	0.1736	-0.3929	0.1778
Dispersion	1.0899	0.0784	0.7738	0.1027	1.1564	0.0837

[1] Base case is a tapered acceleration lane
[2] Base case is more than two upstream lanes

4.2 Recommended SPFs for Non-Isolated Ramp Merge Zones

The final model form for non-isolated ramp merge zones is as follows:

Crashes/year = $\exp^{(\text{intercept+adjustment term})} \text{AADT}^{b}$

where,

AADT is the mainline AADT count

In **Table D,** "Intercept" and b are the estimated model parameters.

Table D. Estimated Parameters for Recommended SPFs for Non-isolated Ramp Merge Zones

Parameter	TOTAL		INJURY		PDO	
	Estimate	Standard Error	Estimate	Standard Error	Estimate	Standard Error
Intercept	-8.4137	1.2122	-7.6103	1.3872	-9.0152	1.2486
b	1.0328	0.1295	0.6988	0.1410	1.0874	0.1342
Intercept adjustment for parallel acceleration lane[1]	-0.8190	0.4546	-0.3069	0.5052	-0.9173	0.4653
Intercept adjustment for diamond interchange ramp[2]	0.4783	0.2832	0.2897	0.3148	0.4950	0.2897
Dispersion	1.1126	0.1894	0.9607	0.2713	1.1409	0.1942

[1] Base case is a tapered acceleration lane
[2] Base case is parclo interchange ramp

Note that the value of the intercept term depends on the type of acceleration lane (parallel or tapered) and on whether the ramp type is diamond parclo loop. When the intercept adjustment is applied its value is simply added to the "Intercept" estimate. If the base case is met the intercept term is not applied.

4.3 Recommended SPFs for Weave Sections

SPFs were developed for weave sections up to 2,500 ft. long, consistent with the HCM specification of the maximum length of a weave section for operational analysis. Similar to the case for non-isolated ramp sections, which has a similar maximum length, crash frequency did not depend on merge zone length. The models are of the form:

Crashes/year = $\exp^{(\text{intercept+adjustment term})}\text{AADT}^{b}$
where
AADT is the mainline AADT count

In **Table E**, "Intercept" and b are the estimated model parameters. Note that the value of the intercept term depends on whether the environment is urban or rural and whether there are two or more than two upstream lanes. When the intercept adjustment is applied its value is simply added to the "Intercept" estimate. If the base case is met the intercept term is not applied.

Table E. Estimated Parameters for Recommended SPFs for Weave Sections with Merge Zone Lengths < 2,500 ft.

Parameter	TOTAL		INJURY		PDO	
	Estimate	Standard Error	Estimate	Standard Error	Estimate	Standard Error
Intercept	-10.7228	2.4140	-12.4927	3.3963	-10.7298	2.4481
b	1.1764	0.2188	1.1247	0.3073	1.1678	0.2218
Intercept adjustment for two upstream lanes[1]	-0.5167	0.2864	-0.2997	0.3910	-0.5417	0.2897
Intercept adjustment for rural location[2]	0.6930	0.3333	1.0350	0.4545	0.6062	0.3339
Dispersion	0.6401	0.1047	0.8655	0.1978	0.6453	0.1062

[1] Base case is more than two upstream lanes
[2] Base case is urban location

4.4 Additional SPFs Investigated

This section documents the SPFs calibrated for the investigation of two exposure-related issues: (1) the use of ramp 1 and 2 volumes as the main exposure independent variable and (2) whether AADT is a key variable in predicting late night crashes.

4.1.1 Use of Lane 1 and Lane 2 Volumes

The HCM procedures logically estimate operational level of service at a ramp-freeway junction based on interactions between the ramp volumes and those in Lanes 1 and 2 (adjacent to the on ramp). It stands to reason that these interactions would strongly influence safety, too. With this mind, Lanes 1 and 2 volumes were estimated using the HCM procedures and SPFs were estimated based on these volumes and compared to SPFs based on the entire directional AADT. The comparison was done for Total crashes at isolated and non-isolated ramps and was based on an examination of the estimated overdispersion parameter. **Table F** shows the models using Lanes 1 and 2 volumes, where V_{R12} is the combined ramp and estimated Lanes 1 and 2 volumes.

Table F. SPFs for TOTAL Crashes at Isolated and Non-isolated Ramp Junctions Using Lanes 1 and 2 Volumes

Parameter	ISOLATED		NON-ISOLATED	
	Estimate	Standard Error	Estimate	Standard Error
Intercept	-1.6900	0.7294	-10.5872	1.4443
B(for $V_{R12\ variable}$)	0.4303	0.0686	1.2710	0.1587
Intercept adjustment parallel acceleration lane[1]	-0.2273	0.1495	-0.7863	0.4740
ln(merge zone length	0.9882	0.6364		
Intercept adjustment for two upstream lanes[2]	-0.5788	0.1599		
Intercept adjustment for diamond interchange ramp[3]			0.5654	0.2899
Dispersion	1.0893	0.0784	1.1633	0.1963

[1] Base case is a tapered acceleration lane
[2] Base case is more than two upstream lanes
[3] Base case is parclo interchange ramp

The comparison of the dispersions parameter with those for Total crashes in **Tables C and D** reveals that the entire directional AADT SPFs are mildly superior to those that used the AADT in Lanes 1 and 2, so it was decided not to further pursue this approach at this time. It is possible that using the actual Lanes and 2 volumes rather than estimates would improve the SPFs developed with these volumes.

4.1.2 Modeling of Late-Night Crashes

The modeling of late-night crashes was also pursued for Total crashes at isolated ramp junctions. However, it was determined that there was no benefit to separating out these crashes for purposes of developing SPFs since models using the daily traffic volume were superior, so this idea was not pursued further. These SPFs are not recommended for application but the ones developed are documented in **Table G** in support of the decision not to pursue this issue further. A comparison of the dispersion parameters for these models with those for SPFs using AADT as the only predictor variable (**Table H**) shows that the latter models are superior, even without considering additional variables. (For both sets of models, segment length was highly insignificant.)

Table G. SPFs for Late Night Crashes at Isolated Ramp Junctions Without AADT Variable

Parameter	TOTAL		INJURY		PDO	
	Estimate	Standard Error	Estimate	Standard Error	Estimate	Standard Error
Intercept	0.4296	0.1105	-1.5399	0.1628	0.2821	0.1119
Adjustment to intercept for rural location[1]	-0.6138	0.1125	-0.6175	0.1896	-0.6161	0.1159
Intercept adjustment for two upstream lanes[2]	-0.8779	0.1329	-0.7367	0.2063	-0.9043	0.1354
Dispersion	0.5877	0.0803	0.5212	0.2315	0.5813	0.1159

[1]Base case is urban location
[2]Base case is more than two upstream lanes

Table H. SPFs for Late Night Crashes at Isolated Junctions Using AADT as the Only Variable

Parameter	TOTAL		INJURY		PDO	
	Estimate	Standard Error	Estimate	Standard Error	Estimate	Standard Error
Intercept	-9.5805	0.5137	-10.5053	1.0192	-9.9950	0.5459
b	0.9130	0.0514	0.8186	0.1005	0.9377	0.0545
Dispersion	0.2731	0.0512	0.2407	0.1794	0.2693	0.0550

5.0 RECALIBRATION PROCEDURE

The SPFs developed apply to similar on-ramp merge areas and weave areas under CDOT jurisdiction during the time period for which the data were collected. It may be desirable at a future time period to recalibrate the models for data from future years. Expected crash frequencies may change over time due to issues such as changes to reporting practices, demographics, statewide safety programs, etc. The desirable recalibration sample size would be such that there are a minimum of 30 to 50 sites of the same site type and at least 100 observed crashes per year.

For the sample, data are collected to apply the SPFs to predict the number of crashes at each site. The ratio of the sums of observations to sum of predictions is used as an estimate of the calibration factor. This calibration factor is then added as a multiplier to the original SPF. This is essentially the same recalibration procedure documented in the Highway Safety Manual for applying an SPF to a different time period or jurisdiction.

It is also logical to recalibrate the overdispersion parameter as this not only indicates how well the recalibrated SPF is fitting the data but can also be used in the empirical Bayes methodology. Procedures with varying complexities for recalibrating the overdispersion parameter are provided below.

5.1 Estimation of Overdispersion Parameter (k) by Maximum Likelihood

The maximum likelihood method estimates the most likely value of the dispersion parameter and is the preferred approach as it is more accurate. The log-likelihood is calculated for a range of possible values of *k*, and the value of *k* with the largest log-likelihood is selected. If there is no such peak in the initial range selected, then a broader range of potential values of *k* is used. It is recommended to initially use values of *k* in increments of 0.5 to get a rough estimate and then to use increments of 0.05 to arrive at the final estimate of *k*.

For each of j = 1 to N sites, the following equations are applied:

a = (1/*k)**LOG((1/*k*)/predicted);

b = ((1/k)+observed)*LOG((1/*k)*/predicted+1);

$$c = \sum_{i=1}^{observed} LOG((1/k)+i-1)$$

Where,

k = the overdispersion parameter

predicted = the number of crashes predicted at site j by the recalibrated crash prediction model

observed = the crash frequency observed at site j

The log-likelihood for k is then calculated as:

$$Log-Likelihood = \sum_{j=1}^{N} a - \sum_{j=1}^{N} b + \sum_{j=1}^{N} c$$

Illustration - As an example, consider a fictitious dataset of sites including the following site j:

Site *j*

Observed crash frequency = 4

Predicted crash frequency = 4.5

Now consider that the analyst has selected a range of *k* from 0.50 to 0.95 in increments of 0.05. To illustrate the use of the above equations we will use the value of *k* = 0.40

a = (1/0.40)*LOG((1/0.40)/4.5) = 2.2447

b = ((1/0.40)+4)*LOG((1/0.40)/4.5+1) = 1.2473

c = LOG(1/0.40+1-1)+ LOG(1/0.40+2-1)+ LOG(1/0.40+3-1)+ LOG(1/0.40+4-1) = 2.3356

Similar calculations are then performed for each site and the log-likelihood calculated. For *k* = 0.40, the table below shows that the log-likelihood is estimated at 2705.

The log-likelihood is calculated for all possible values of *k* selected. As can be seen below, there is a peak value of the log-likelihood when *k* = 0.75 and the value of log-likelihood is 2718. Thus, the estimated value of *k* is 0.75.

k	Log-Likelihood
0.40	2705
0.45	2707
0.50	2708
0.55	2711
0.60	2712
0.65	2714
0.70	2716
0.75	2718
0.80	2715
0.85	2713
0.90	2708
0.95	2706

5.2 Estimation of Overdispersion Parameter (k) by Linear Regression

Step 1: For each site, use the recalibrated crash prediction model to estimate the expected number of crashes (P). Also compute P^2.

Step 2: For each site, determine the value of the squared residual (SR):

$$SR = (P - Crash\ count)^2$$

Step 3: Subtract the value of P from the squared residual (SR). This gives an estimate of $P^2 * k$:

$$[\text{Estimate of } P^2 * k\ ] = SR - P$$

Step 4: Fit a linear model through the origin with $P^2 * k$ as the dependent variable and P^2 as the independent variable. An ordinary least squared regression procedure such as can be executed in Microsoft Excel should suffice.

Step 5: The calibrated slope of the regression line is an estimate of k.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The development of SPFs for the three categories of ramp-freeway merge zones, classified as isolated, non-isolated and weaves, was successful. Separate SPFs were developed for Total, FI (fatal+non-fatal injury) and PDO crashes. The SPFs contain logical variables with intuitive directional effects.

If crash predictions are desired for two or more severities out of Total, FI and PDO, it is recommended to use the SPFs for PDO and FI crashes directly. If one of the desired severities is Total, then add the estimates for PDO and FI. The reason is that at some rare extremes of the variables included in the SPFs the estimate for PDO or FI could be greater than that of Total.

All of the SPFs have AADT downstream of the ramp as the main exposure variable. Consistent with the operational analysis in the HCM, estimated Lanes 1 and 2 volumes (adjacent to the ramp) was also considered instead of the entire directional mainline AADT, but that investigation revealed that the entire mainline AADT was a better predictor of crashes in the ramp influence area as defined for this project.

AADT on the ramp was also considered as an additional exposure variable since, logically, expected crash frequency should increase with an increasing ramp AADT. In the modeling, however, the ramp AADT variable was never statistically significant, and the effect was always small. This finding is consistent with the proposed HCM SPFs for ramp merges for which there was only a small influence of ramp AADT for FI crashes and no effect for PDO crashes.

The modeling of late-night crashes was also pursued for isolated ramp junctions under a hypothesis that AADT may not be a key variable in predicting these crashes. However, it was determined that there was no benefit to separating out these crashes for purposes of developing SPFs, so this idea was not pursued further. A comparison of the dispersion parameters for these models with those for SPFs using AADT as the only predictor variable showed that the latter models are superior, even without considering additional variables.

Budget constraints limited the sites pursued for this project to those where a ramp AADT was available. It is recommended that data for additional sites be collected as they may become available. Such data can be used to screen the entire network and ultimately to enhance the developed SPFs. Additionally, as more years of crash and traffic data become available, these data can be added to the database to continually update information. The SPFs can be recalibrated to apply to these additional years of data using a procedure documented in the report. When several additional years of data and sites are available, it may be desirable to calibrate a new set of original SPFs.

APPENDIX A – SAMPLE PROBLEMS

This appendix provides several potential applications of the Safety Performance Functions (SPFs) developed for merge zones. These applications represent the state-of-the-art in road safety management and are covered in the Highway Safety Manual (HSM), First Edition. Four applications are presented:

1. Comparison of Observed Crash History to Expected
2. Ranking of Several Locations for Further Investigation and/or Safety Improvement
3. Benefit-Cost Evaluation of Contemplated Safety Improvements
4. Evaluation of the Safety Effectiveness of Implemented Safety Improvements

Sample Problem 1 – Comparison of Observed Crash History to Expected

In this application, a single site is being analyzed to assess whether it is performing better or worse in terms of total crash frequency than would be expected for similar sites. The approach compares the site's expected crash frequency as determined by the empirical Bayes (EB) procedure to the estimate for an average site as determined solely by the appropriate SPF.

The characteristics of this site are:

- Site Type = isolated ramp
- Parallel Merge Lane
- Number of Lanes Upstream = 2
- Merge Zone Length = 0.81 miles
- Mainline AADT = 4,930
- 5 Year Total Crash History, X, = 105

Because the site type is an isolated ramp merge zone, the SPFs from Table 3 are applicable.

Step 1: Estimate the expected crash frequency per year for similar sites using the SPF for total crashes in Table 3.

Crashes/year = (merge zone length) $\exp^{(\text{intercept+adjustment for parallel vs taper lane + adjustment for lanes upstream of merge})} AADT^{b}$

Crashes/year $= (0.81) \exp^{(-1.8371-0.2189-0.3844)}(4{,}930)^{0.4250}$

$= 2.62$

<u>Step 2</u>: Estimate the empirical Bayes (EB) weight.

$$w = \frac{1}{1+knP} = \frac{1}{1+1.0899(5)(2.61)} = 0.0657$$

where,

k is the overdispersion parameter of the SPF from Table 3

n is the number of years of crash data available

P is the estimate from Step 1

Note that since 5 years of crash history are being used, the weight, w, is relatively small, meaning more weight will be given to the observed crash count than to the SPF estimate. If fewer years of crash data were available or desired for consideration, then more weight would be given to the SPF estimate. This recognizes that with fewer years of data, regression-to-the-mean is of increasing concern.

<u>Step 3</u>: Estimate the EB expected crash frequency for the 5 year period.

$$m = wn(P) + (1-w)(X) = 0.0657(5)(2.62) + (1-0.0657)(105) = 99$$

The EB estimate of crashes during the 5 year period is 99 crashes.

Step 4: Compare the EB estimate from Step 3 to the SPF estimate from Step 1.

A comparison of the expected crash frequency of this site for the 5 year period is 99, which is much higher than the SPF estimate of (5*2.62) = 13 crashes. Compared to other isolated merge zones with the same length, AADT and other characteristics represented by the SPF, this site is experiencing many more total crashes. Based on this information, further investigation of the site would seem justified to assess whether there is, indeed, a safety concern that may be remedied.

Sample Problem 2 – Ranking of Several Locations for Further Investigation and/or Safety Improvement

Because resources do not permit all locations to be investigated to assess their need for safety improvements, a methodology for prioritizing locations is required. This is referred to as Network Screening. The Highway Safety Manual documents several approaches, with the recommended approach based on the EB estimates of expected crash frequency as calculated in Sample Problem 1. The EB estimate for all sites would be determined using the appropriate SPF for each site as illustrated in Sample Problem 1. Then all sites would be ranked in descending order by the EB estimate. An alternate method is to rank in descending order by the difference between the EB estimate and the SPF estimate.

The table below shows the ranking for the top 4 sites of a fictitious dataset.

Site ID	Average Crash Frequency/year	SPF Estimate of Crashes/year	EB Estimate of Expected Crashes/year	Rank
83	23.2	6.5	21.8	1
104	21.0	5.0	19.8	2
15	26.0	2.5	16.6	3
62	15.6	2.3	10.6	4

Sample Problem 3 – Benefit-Cost Evaluation of Contemplated Safety Improvements

The third application is conducting a benefit-cost analysis of a contemplated safety improvement. Consider again the site from Sample Problem 1. The EB estimate of expected crash frequency was 99 for the 5 year period, or 19.8 crashes per year. A countermeasure for improving safety is being considered for which the Crash Modification Factor (CMF) is 0.85.

Step 1: Estimate the expected reduction in crashes.

% Reduction = (1-CMF)(EB estimate) = (1-0.85)(19.8) = 2.97 crashes/year

Step 2: Compare the benefits and costs.

The crash reduction benefits are estimated as 2.97 crashes/year. The economic benefit of this crash reduction is estimated by applying the jurisdiction's crash cost estimate.

The economic costs should reflect any capital and maintenance costs associated with the countermeasure, as well as any other factors required to be considered.

Sample Problem 4 – Evaluation of the Safety Effectiveness of Implemented Safety Improvements

In this application, a safety countermeasure has previously been implemented at a location where it was determined that a safety problem existed. We will again use the site from Sample Problem 1 to illustrate.

The estimate of expected crash frequency in the 5 year period was 99, or 19.8 crashes per year. Now, there is an additional 3 years of observed crashes following installation of the safety countermeasure. In this 'after period,' the average AADT has increased to 5,500 and the observed total crash frequency is 45.

Step 1: Estimate the EB estimate of expected crash frequency prior to treatment.

This step has been done in Sample Problem 1. The EB estimate is 99 for the 5 year before period.

Step 2: Use the appropriate SPF to estimate the number of crashes in the after period for similar sites. The calculations are the same as those for Step 1 in Sample Problem 1, but now the AADT has increased to 5,500 and the after period is 3 years.

$$\text{Crashes/year} = (0.81)\exp^{(-1.8371-0.2189-0.3844)}(5{,}500)^{0.4250}$$
$$= 2.74$$

This after period annual SPF estimate is used, along with the before period annual SPF estimate, to account for the differences in AADT and duration of the before (5 years) and after (3 years) periods as follows:

$$r = SPF_{after}/SPF_{before} = 2.74(3)/2.62(5) = 0.63$$

Step 3: Estimate the EB estimate of after period crashes had no countermeasure been applied and the variance of this estimate. The weight, w, as calculated in Sample Problem 1 is used here again.

$$EB_{after} = EB_{before}{*}r = 99(0.63) = 62.4$$

$$Var\{EB_{after}\}=r^2 EB_{before}{*}(1-w) = 0.63^2(99)(1-0.0657) = 36.71$$

Step 4: Calculate the index of effectiveness, theta, and its variance.

$$\text{theta} = (Crashes_{after}/\ EB_{after})/(1+\ Var\{EB_{after}\}/\ EB_{after}{}^{2})$$
$$=(45/62.4)/(1+36.71/36.71^2)$$
$$= 0.70$$

$$Var\{theta\} = theta^2(1/\ Crashes_{after}+\ Var\{EB_{after}\}/\ EB_{after}{}^{2})/[(1+\ Var\{EB_{after}\}/\ EB_{after}{}^{2})^2]$$
$$= 0.70^2(1/45+36.71/62.4^2)/[(1+36.71/62.4^2)^2]$$
$$= 1.52$$

The standard deviation of theta is then $1.52^{0.5} = 1.23$

Theta is referred to as the Crash Modification Factor. The percent change in crashes is calculated as 100(1−theta). Thus, a value of θ = 0.70 with a standard deviation of 1.52 indicates a 30 percent reduction in crashes with a standard deviation of 152 percent. The standard deviation is quite high, which reflects the reality that the results are based on only one site.

If multiple sites were available for evaluation, Steps 1 to 3 would be conducted for each site. Then, the estimates of EB_{after}, $Var\{EB_{after}\}$ and the observed crash frequency in the after period are summed over all sites and these values used in Step 4.