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Evaluation of Roadside Treatments to Mitigate Roadway Departure Crashes: Technical Report

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16. Abstract
This report presents findings from a research project that sought to provide a systemic framework to identify high-risk locations for roadway departure crashes and applicable countermeasures for implementation. The products of the research project are intended to assist Texas Department of Transportation (TxDOT) districts with selecting projects that address roadway departure crashes proactively (such as by improving guardrails and barriers, or by safety-treating roadside fixed objects) as opposed to reactively (by basing projects on crash history only). Additionally, this report provides updated work codes to assist TxDOT in better prioritizing projects, making a more optimal use of limited resources, and maximizing benefits derived from projects implemented as a result. For this effort, a select number of work codes were evaluated, and updates to their associated crash modification factors were developed based on available data on past Highway Safety Improvement Program projects in the state.

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DISCLAIMER

This research was performed in cooperation with the Texas Department of Transportation (TxDOT) and the Federal Highway Administration (FHWA). The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of FHWA or TxDOT. This report does not constitute a standard, specification, or regulation.

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CHAPTER 1. PROJECT OVERVIEW

The research project discussed in this report sought to provide the Texas Department of Transportation (TxDOT) with a systemic framework to identify high-risk locations for roadway departure crashes and applicable countermeasures for implementation. The products of this research project are intended to help TxDOT districts select projects that address roadway departure proactively (for example, by improving guardrails and barriers or by safety-treating roadside fixed objects) as opposed to reactively (i.e., based on crash history only). Additionally, this project sought to provide updated work codes to assist TxDOT in better prioritizing projects, making a more optimal use of limited resources, and maximizing benefits derived from projects implemented as a result.

This report is structured in six chapters that document and summarize the findings from this research effort. Chapter 1 is an introduction to the document and an overview of the report structure. Chapter 2 provides a literature review on the topic of roadway departure crashes and countermeasures. Chapter 3 documents the proposed study approach and subsequent data collection efforts. Chapter 4 documents the systemic approach applied to a Texas statewide representative sample of roads and the results from that effort. Chapter 5 documents the safety evaluation of select roadway departure crash countermeasures and recommended updates to the corresponding work codes. Chapter 6 summarizes the conclusions and recommendations based on the research effort.

CHAPTER 2. LITERATURE REVIEW

This chapter documents the literature review performed at the beginning of the research project. The chapter consists of three parts: (a) a review of literature on past works that studied countermeasures for roadway departure crashes; (b) a review of the current status of the systemic approach to identifying locations of promise to deploy safety countermeasures, with an emphasis on addressing roadway departure crashes; and (c) conclusions from the literature review and further steps.

LITERATURE REVIEW ON ROADSIDE COUNTERMEASURES

The Federal Highway Administration (FHWA) defines roadway departure as a crash that occurs after a vehicle crosses an edge line, centerline, or otherwise leaves the travel way. From 2014 to 2016, an average of 18,779 fatalities resulted from roadway departure, which is 53 percent of all traffic fatalities in the United States. Texas reported 61,973 roadway departure crashes in 2016 alone. The Texas Strategic Highway Safety Plan developed in 2017 identified roadway departure crashes as one of the seven emphasis areas for the next 5 years. To effectively prevent roadway crashes, FHWA promotes countermeasures deploying three types of strategies: (a) keep vehicles on the roadway, (b) provide for safe recovery, and (c) reduce crash severity.

The following subsections summarizes current knowledge about the safety, operational, and economic effectiveness of different countermeasures that address roadway departure crashes.

Overview of Side-Slope Flattening for Safe Recovery

Roadways with steep side slopes have seen many single-vehicle roadway departure crashes. To reduce the severity and the number of roadway departure crashes, TxDOT requires the embankment side slopes to be flattened to 6:1 or flatter. As per the state’s work code manual for the Highway Safety Improvement Program (HSIP), flattening of slide slopes should yield a crash reduction factor of 46 percent (Rawson, 2015). The following sections document the research carried out to evaluate the benefits/effectiveness of side-slope flattening in reducing the frequency and severity of roadway departure crashes.

Safety and Operational Evaluation of Side-Slope Flattening

This section summarizes the data, methods, and conclusions of the most relevant studies evaluating the safety performance of side-slope flattening.

Zeeger et al. (1995) investigated the effect of side slopes based on the field-measured side slope on 1,776 mi of roadway in three states: Alabama, Washington, and Michigan. They conducted statistical testing along with log-linear modeling to determine the interactive effects of roadway features, such as flattening of side slopes, on roadway departure crashes. The researchers reported that steeper side slopes were found to increase rates of single-vehicle and rollover crashes. Compared to steeper slopes, side slopes in the range of 3:1 to 7:1 or flatter were found to have lower single-vehicle crash rates ranging from 2 percent to 27 percent.

Allaire et al. (1996) conducted a before-after study to evaluate the validity of design guidelines for flattened side-slope sections of highways in Washington State. Run-off-road (ROR) collision data were also collected for evaluating the benefit-cost (B/C) analysis of the roadway treatment. To determine if the flattened sections experienced fewer ROR collisions, a total of 750 contracts that were completed in the calendar years 1986 through 1991 were identified for study. As-built drawings, available design reports, quantity summaries, cross-section plans, plan views, and conversion equations were examined to determine the extent of side-slope flattening and other roadside safety features. The results showed that the overall percentage reduction in collisions due to slope flattening exceeded 15 percent, which was the default value used at the time in B/C analysis.

Economic Effectiveness Evaluation of Side-Slope Flattening

Schrum et al. (2014) conducted research to investigate the real-world benefits of slope flattening in terms of B/C analysis. Crash data were collected over a 7-year period between 2000 and 2006 in the state of Ohio to correlate crash severity with slope embankments. Highways were modeled using the Roadside Safety Analysis Program (RSAP). RSAP models were simulated for various parameters that influenced the crash costs and for varying steepness of embankment slopes. The simulated crash costs were used to evaluate the B/C analysis of slope flattening. The researchers found that on freeways and urban arterial highways, slopes should be no steeper than 1V:3H, and the benefit of flatter slopes was minimal. On rural arterial highways, the slopes should be no steeper than 1V:4H (with a reduction in crash costs greater than 65 percent), and the benefit of flatter slopes was minimum. Local highways were found to benefit the most from slope flattening. It was found that the steepest slope should be 1V:3H, but the slope should be made as flat as possible because the crash costs continued to decrease as the slope was flattened.

Crash Modification Factors of Side-Slope Flattening

Jurewicz et al. (2014) conducted a project to develop a safety management framework for Australia and New Zealand with updated advice on hazard management and treatment selection to focus on minimizing the fatal and serious injuries resulting from ROR crashes. Undivided rural data (sourced from a site investigation, digital video recordings, and a literature review) in the state of Victoria were analyzed for speed limits of 100 km/h at 2900 km to determine relative ROR casualty crash risk for various slopes. The researchers found that ROR crash likelihood more than doubled for steep roadsides (1:3.5 or steeper) compared to flat roadsides (1:6 or less). The crash severity, on the other hand, was shown to decrease with increasing side slope. The most effective treatment was found to be flattening side slopes from 1:4 to 1:6, and from 1:3 to 1:6, both of which had a crash reduction factor (CRF) of 24 percent, or a crash modification factor (CMF) of 0.76.

Overview of Using Guardrails for Reducing Crash Severity

A guardrail is a safety barrier intended to shield a motorist who has left the roadway. Guardrails are meant to lessen the severity of crashes. Guardrails have been widely used in all motorized countries to reduce the consequences of crashes in which vehicles run off the road or cross the road on divided highways. It is a general rule to install the guardrail at sections where the consequences of striking the guardrail are considered less serious than the consequences of striking the guarded object. As per the FHWA policy, the roadside safety hardware installed on the National Highway System should follow the crash testing and evaluation criteria contained in the *Manual for Assessing Safety Hardware* or National Cooperative Highway Research Program (NCHRP) 350. The following sections summarize the literature available to evaluate the effectiveness of guardrails in reducing the severity of roadway departure crashes.

Safety and Operational Evaluation of Guardrails

Elvik (1995) conducted research that reports the results of a meta-analysis of 32 evaluation studies that quantified the safety effects of median barriers and guardrails along the edge of the road. The author included 232 estimates in his meta-analysis. Elvik concluded that guardrails are expected to reduce the chance of fatal injury by 45 percent, given that a crash has occurred. The chance of sustaining personal injury is expected to decrease by 50 percent. The report also points out that the effects of guardrails on crash frequency have been less extensively studied than the effects on crash severity.

An internal report by the Missouri Department of Transportation (MoDOT) (Chandler, 2007) evaluated the benefits of installing median cable barriers on Interstate Highway 70. Chandler (2007) reported that on Interstate 70, the installation of 179 mi of median cable barrier on the freeway had nearly eliminated cross-median roadway deaths. The number of cross-median fatalities on Interstate 70 had been increasing, reaching a peak of 24 motorcyclists killed in 2002. Following the installation of the cable barrier, only two cross-median fatalities occurred in 2006, a 92 percent decrease.

Park et al. (2016) conducted a before-after study to evaluate the safety effects of adding specific types and combinations of roadside barriers on freeways for different crash types and severity levels based on different vehicle type, driver characteristics, weather conditions, and time changes. The road geometry data were collected from the Florida Department of Transportation for a period of 9 years (2003–2011) from the Roadway Characteristic Inventory. A total of 147 segments with a total length of 67,178 mi were identified. A W-beam guardrail was installed in 127 sites, while 20 sites had implemented concrete barriers. The crash records were obtained from the crash analysis reporting system for the 4-year period before (2003–2006) and 4-year period after (2008–2011). Two observational before-after analyses—empirical Bayes (EB) and full Bayes (FB)—were used to estimate CMFs. The researchers differentiated by vehicle size (passenger and heavy), driver age (young, middle, and old), weather condition (normal and rain), and time period (daytime and nighttime) in their analysis. Park et al. found that the addition of roadside barriers was effective in reducing severe crashes for all types and ROR crashes. On the other hand, it was found that roadside barriers tended to increase all types of crashes for all severities. This finding implies that the treatment might increase the total number of crashes, but it might be helpful in reducing injury and severe crashes. It was also found that the CMFs for injury and severe ROR were lower for heavy vehicles than passenger cars. Furthermore, the results indicate that guardrails seem to be more associated with reduced injury and severe ROR crashes for middle age and older drivers than for younger drivers. It was also found that the safety effects of the treatment were higher for injury and severe ROR crashes during nighttime than daytime. Last, the CMFs were lower for severe ROR crashes in rain conditions than normal conditions. A major limitation of the study is the relatively small sample size. The researchers recommended studying more variations of safety effects, including pavement conditions, seasonal difference, and so forth, as well as conducting a more detailed categorization of barrier types.

Li and Park (2017) conducted research to evaluate the effect of in-service guardrail systems in reducing fatal and severe injury crashes on freeways. The study also evaluated the effects of different guardrail systems (strong-post guardrails, weak-post guardrails, and cable guardrails) in reducing crash severity. A total of 6,415 single-vehicle roadway crashes that occurred on interstate systems from 2010–2014 within the Richmond District of the Virginia Department of Transportation (VDOT) were identified from Virginia's statewide crash database. A total length of 366 mi of guardrail system installed on 535 directional miles of interstate road segments within the Richmond District were collected from the VDOT database. Each pair of roadway departure crashes hitting the guardrail had been associated with the corresponding guardrail inventory to determine the guardrail information, such as type. A binary logistic model was developed to identify factors such as guardrail presence, vehicle type, and driver safety behavior, and statistical analyses were conducted to compare severities of roadway crashes (for example, hitting vs. not hitting the guardrail, or the impacts of different types of guardrails on crash severity). The researchers reported that hitting an in-service guardrail was expected to reduce the probability of a roadway crash resulting in a fatal or severe injury by about 45 to 50 percent, compared to a roadway crash involving not hitting a guardrail. The results indicate that strong-post W-beam guardrails result in significantly more fatal and severe crashes compared to low-tension cable systems. The findings of the report provide practical values for developing a data-driven and risk-based guardrail investment and cross-asset resource allocation strategies.

Economic Effectiveness Evaluation of Guardrails

Gates et al. (2006) conducted a study to determine the average daily traffic (ADT) threshold at which installation of a bridge approach rail on low-volume roads is cost effective based on reductions in crash severity. Bridge and crash data from Minnesota were used in the study. A dataset from a total of 398 bridges was obtained (155 with approach guardrails and 243 without approach guardrails). Crash data were obtained from the Minnesota state crash database, and the final crash sample size consisted of 96 crashes: 47 with bridges at the approach guardrail and 49 at bridges without the approach guardrail. Logistic regression and chi-square tests were used to analyze the characteristics of 96 ROR crashes in Minnesota over a 15-year period. B/C analysis was subsequently conducted to determine the cost effectiveness of a bridge approach rail. The researchers found that 96 Minnesota locations showed significantly lower rates of severe crashes at all bridges where an approach rail existed, except for those with very low ADT. The B/C analysis showed that the approach guardrail was cost effective (B/C >1) at all bridges except those with ADTs smaller than 400 vpd, and the treatment became increasingly more cost effective with increases in ADT. Overall, the B/C ratio ranged from 3.12 to 4.35 depending on guardrail installation/maintenance cost.

Effect of Guardrails on Driver Perception for Crash Prevention

A simulation-based study by Bassat and Shinar (2011) tested the combined effects of three roadway design elements—shoulder width, guardrail existence, and roadway geometry (curvature)—on objective driving measures (speed and lane position) and subjective driving measures (perceived safe driving speed and estimated road safety). There were 21 driver participants in the experiment with a driving simulator. The simulations used a counterbalanced experimental design that allowed the researchers to examine the effects of three roadway effects simultaneously while controlling all the other possible factors. The experimental design included

30 possible combinations of the manipulated roadway design factors. Bassat and Shinar reported a significant effect of roadway geometry on both objective and subjective measures. The shoulder width had a significant effect on the actual speed, on lane position, and on perceived safe driving speed, but only when a guardrail was present. The findings suggest that a guardrail heightens the sense of security that wide shoulders are designed to provide, illustrating the perceptual role of a guardrail in defining perceived safety margins. When a guardrail is absent, the width of the shoulder loses much of its benefits and effects on driving behavior. The report concludes that guardrails can serve not only as a post-crash injury reduction measure but also as a crash prevention device since guardrails have a very strong perceptual effect on the width and safety margin that the shoulder provides, to the point that they affect driving speeds, and thus driving safety.

Overview of Median and Impact Attenuators

Crash cushions (impact attenuators) are used to reduce the severity of an impact with a fixed, narrow object. This is usually accomplished by absorbing the kinetic energy of the vehicle. Crash cushions are ideal for fixed objects that cannot be removed, relocated, or shielded by longitudinal barriers (AASHTO 2011). While crash testing provides an objective basis for evaluating the safety effectiveness, different crash cushions are observed to vary in terms of operations and geometry. It is reasonable then to assume that different crash cushions would have varying abilities to mitigate occupant risk. This section documents the effectiveness of crash cushions and median barriers, per the current safety literature.

Safety and Operational Evaluation of Crash Cushions

Rune Elvik (1995) conducted research that reports the results of a meta-analysis of 32 evaluation studies that have quantified the safety effects of median barriers, guardrails, and crash cushions along the edge of the road. His meta-analysis incudes 232 estimates. Elvik concluded that crash cushions appear to reduce both crash rate and crash severity. However, the numerical estimates of the effects of crash cushions were particularly uncertain due to the methodological shortcomings of the evaluation studies.

Zou et al. (2014) conducted a study to understand the safety performance of roadway and median barriers. Single-vehicle crashes were studied to compare the risk of injury among different hazardous events, including rolling over and striking the road barriers (guardrails, concrete barrier walls, and cable barriers). A total of 2,124 single-vehicle crashes (3,257 occupants) that occurred between 2008 and 2012 on 517 pair-matched homogenous barrier and non-barrier segments in the state of Indiana were analyzed. The data needed for the study included barriers data, roadway data, traffic volumes, single-vehicle crash data, and occupant injury records for the analyzed period of 2008–2012. Data were collected from the Indiana Department of Transportation Work Management system. Google Earth images were also used to calculate barrier offsets. A binary logistic regression model was estimated for vehicle occupants. The model was developed to estimate the difference in the risk of vehicle occupant injury between hitting a barrier and hitting a dangerous roadside object. The observations were categorized into two severity levels: injury level (fatality, incapacitating and non-incapacitating) and non-injury level (possible injury and property damage only). The modeling results revealed that hitting a barrier is associated with lower risk of injury than a higher hazard event (hitting a pole, rollover, etc.). The odds of injury are expected to be 43 percent lower when striking a guardrail instead of a median concrete barrier offset 15–18 ft and 65 percent lower when striking a median concrete barrier offset 7–14 ft. The odds of injury when striking a near-side median cable barrier are 57 percent lower than the odds for a guardrail face. The reduction for a far-side median cable barrier is 37 percent. Zou et al. reported that installing median cable barriers on both sides of the median to reduce their lateral offset is beneficial for safety. These researchers recommended high-tension cable barriers and W-beam guard rails as a viable median barrier alternative for a wide median (50 ft and wider), whereas both concrete and barrier walls and W-beam guardrails could be considered in a narrow median (15 ft).

A study conducted by Burbridge et al. (2015) used data derived from crash testing of 11 re-directive crash cushions as the base input to a numerical procedure for calculation of occupant risk indicators—occupant impact velocity, occupant ride-down acceleration, and longitudinal acceleration severity index—for a range of simulated impacting vehicles (mass 800 kg to 2,500 kg) impacting each crash cushion at a range of impact speeds (18 m/s to 32 m/s). The objective of the study was to improve the knowledge about the device over a range of impact parameters and to help determine the crash cushion most suited to a particular application. The results from the study indicate that occupant risk varies with the varying types of crash cushions being impacted. In other words, devices that satisfy the same test protocol may not mitigate risk equally. In terms of occupant severity, results indicate that variation in crash cushion performance during end-on impacts improves with decreasing impact mass, and to a lesser extent with decreasing impact speed. The results also indicate that impact severity increases with decreasing mass of the impacting vehicle. Burbridge et al. concluded that different devices may perform differently for the end-on impact configuration.

Economic Effectiveness Evaluation of Crash Cushions

A study was conducted by Schrum et al. (2014) to determine the B/C ratios for each cushion category in a wide range of roadway and roadside characteristics using the probability-based tool Roadside Analysis Program. Crash cushions were categorized in three different categories: redirecting with repair costs greater than $1,000 (RGM), redirecting with repair costs less than $1,000 (RLM), and non-redirecting sacrificial (NRS). To estimate the cost of all crash cushions used in this study, crash cushion systems were examined to understand dimensions and associated costs with each system via manufacturer product sheets and survey questionnaire responses received from the following states' DOTs: Kansas, Minnesota, and Wisconsin. The state DOTs provided information on the average installation cost, average crash repair cost, and average regular maintenance cost. B/C analyses were conducted in two ways: (a) the index method, to compare crash cushions to only the baseline option; and (b) the incremental method, to ascertain the optimal cost-effective option. Schrum et al. reported that only RGM and RLM systems were cost effective for freeways and divided rural arterials, but all three categories were cost effective against the unprotected condition on undivided rural arterials and local roads. RLM systems would be cost effective at locations that experience higher crash frequencies, while RGM crash cushions would be a more feasible option at locations with moderate or low crash frequencies. NRS crash cushions are generally less expensive but require total replacement after a crash has occurred, which may be impractical at high traffic volume locations. The do-nothing alternative would only be recommended on locations where there is a very large crash cushion offset and/or very low traffic volume.

Overview of High-Friction Surface Treatment on Curves

Pavement friction plays important roles in traffic safety, especially under wet, icy, or slippery conditions. High-friction surface treatment (HFST) is a treatment that uses a polymer binder to restore and/or maintain roadway surface friction at existing or potentially high-friction-related crash areas (Cheung, 2018). Since the early 2000s, HFST has been widely used by transportation departments, with nearly all state DOTs in the United States having applied HFST on roadways, typically at horizontal curves.

In the FHWA *Low-Cost Treatments for Horizontal Curve Safety*, skid-resistive pavement surface treatment, which is quite similar to HSFT, is listed as an effective treatment for mitigating curve-related crashes (Albin et al., 2016). It has been reported that the skid-resistive pavement surface in New York State reduced wet-road crashes by 50 percent and total crashes by 20 percent (Albin et al., 2016).

Safety and Operational Evaluation of HFST

Brimley and Carlson quantified the potential safety benefits of applying HSFT at horizontal curves (Brimley, B.; Carlson, P. 2012). These researchers reviewed the application of HSFT as well as the observed crash reductions. Judging by simple comparisons, this research showed that crashes declined at all sites after the installation of HFST in Florida, Washington, Pennsylvania, Kentucky, and Wisconsin. Brimley and Carlson concluded a 20% to 30% reduction in all crashes and a 50% reduction in wet-weather crashes for general HFST applications.

Similarly, Gan et al. reviewed studies pertaining to Crash Reduction Factors (CRFs) for pavement improvements (Gan, Shen and Rodriquez 2005). They found that resurfacing a curve with a skid-resistant overlay is expected to reduce all crashes by 10 percent to 24% and wet pavement crashes by 51%. These findings are generally in line with (Harkey, et al. 2008).

Merritt et al (Merritt, Lyon and Persaud 2015) estimated the crash modification factor (CMF) for HFST. The researchers collected data at 27 ramps and 43 curves from multiple states: Kansas, Kentucky, Michigan, Montana, South Carolina, Tennessee and Wisconsin. Naïve before-after study showed that wet-road crashes appeared to reduce by 63% and total crashes by 37 percent after the installation of HSFT. Control-group analysis revealed the two types of crashes declined by 52 percent and 24 percent, respectively. The researchers could not apply the EB method due to insufficient treatment and reference site data.

HFST and Crash Severity

Recently, Musey et al. (2017) evaluated effectiveness of HSFT in reducing the severity of crashes. The researchers collected crash data at 74 sites where HSFT was implemented on curvature roadways in Pennsylvania. Simple before-after studies were conducted by level of curvature (i.e., low, moderate, and high curvature). Results indicated that the installation of HFST appeared to reduce the number of crashes by at least 75 percent for each degree of curvature and each crash severity. Most importantly, fatal crashes were completely eliminated at all the study sites in the after period. It is important to note that the researchers applied simple before-after (i.e., naïve before-after) studies, which could be susceptible to regression-to-the-mean bias.

Overview of Pavement Markings

Pavement markings have been used on roadways for decades. Safety analysts have been continuously assessing the safety effects of markings in reducing crashes since the 1950s. About 100 CMFs reported in the CMF Clearinghouse are associated with pavement markings (FHWA n.d.-c).

Safety and Operational Evaluation of Pavement Markings

Elvik and Truls (2004) reviewed the evaluations conducted from the 1950s to the 1990s on the different types of roadway markings. The researchers conducted a meta-analysis on existing results and concluded that the normal edge-line markings increased both injury and property damage only (PDO) crashes by 3 percent. Wider edge-line markings reduced injury crashes by 5 percent and increased PDO crashes by 1 percent. It is worth mentioning that all the results were not statistically significant at the 95 percent confidence level.

NCHRP Synthesis 306: Long-Term Pavement Marking Practices: A Synthesis of Highway Practice (Migletz & Graham, 2002) describes installing pavement markings as an effective way of reducing single- and multiple-vehicle crashes that occur during darkness or nighttime, and particularly under conditions of wet pavements, rain, and fog. In the synthesis, the authors cited an unpublished work and concluded that crashes appeared to decrease by an average of 11 percent after installing pavement markings.

Sun and Tekell (2005) analyzed before and after crash rates on rural roadways with narrower lanes in Louisiana. In a follow-up study by Sun and Das (2012), the researchers analyzed 3 years before and 1 year after the implementation of edge lines on rural two-lane highways in Louisiana using the EB methodology. The results imply that on average, crashes appeared to reduce by 17 percent after installing edge-line markings on rural two-lane narrow highways.

Miles et al. (2010) and Park et al. (2012) evaluated the safety effects of wider edge lines on rural two-lane highways in three states: Kansas, Michigan, and Illinois. For the Kansas data, the researchers analyzed 2,801 segments that had implemented wider edge lines from 2005 through 2008 with the EB before-after method. For the Michigan data, the researchers utilized a cross-sectional regression method with consideration of time series. The Michigan data included 8 years of crash data at 253 segments. For the Illinois data, the researchers applied cross-sectional analysis. Although the developed CMFs were not identical, crashes appeared to decrease after the implementation of wider edge lines in all three states. Overall, total crashes reduced by 17.5 percent to 27.4 percent, and single-vehicle crashes reduced by 28.7 percent to 37.0 percent.

Crash Modification Factors for Pavement Markings

Lyon et al. (2017) estimated CMFs for curve warning pavement markings at curves. The researchers analyzed data in four states—Iowa, Kansas, Missouri, and Pennsylvania—with the EB before-after methodology for different crash types and severity level. It was found that total and night cashes declined after the installation of pavement markings at curves, and the result was statistically significant at the 95 percent confidence level. However, CMFs were not provided due the limited crash number in the after period.

Overview of Rumble Strips/Rumble Stripes

Rumble strips have been widely used as an effective countermeasure for reducing roadway departure crashes by alerting drivers with sound and vibration. Many studies have been conducted by safety analysts on the safety effectiveness of rumble strips in reducing crashes. A synthesis study by the Texas A&M Transportation Institute focused on the development of a rumble strip work code (Walden et al., 2015) to be used in the state's HSIP. A comprehensive review on the effectiveness of various types of rumble strips from that report is summarized in the following section. Summaries of additional, more recent studies are included as well.

Safety and Operational Effectiveness of Shoulder Rumble Strips

Perrillo (1998) conducted a safety effectiveness evaluation of shoulder rumble strips on freeways in New York State. Between 1992 and 1996, shoulder rumble strips were installed on approximately 2,159 shoulder-miles of thruway, a tollway privately owned by the New York State Thruway Authority. Crash data from 3 years (from 1991 to 1993) before the installation and 2 years after (1996 and 1997) on those segments were collected. The result revealed that the CRF for installing shoulder rumble strips on freeways is 79 percent (Perrillo, 1998). This result is included in the first edition of the *Highway Safety Manual* (HSM) (Table 13-45) (AASHTO 2010).

Smith and Ivan (2005) evaluated the safety benefits of shoulder rumble strip installation in Connecticut using a cross-sectional method. Approximately 1,000 mi of segments throughout Connecticut were selected, including sections with and without rumble strips. Shoulder rumble strips were installed on part of these segments between September 1996 and May 1996. Crash data from 36 months before the installation and after were collected. Several variables (e.g., illumination, speed limit, number of lanes, etc.) were also considered in the cross-sectional study. Regression analysis indicated installing milled shoulder rumble strips appeared to reduce single-vehicle run-off-road (SVROR) crashes by 34 percent.

Torbic et al. (2009) collected more than 600 mi of undivided two-lane highways from three states (i.e., Minnesota, Missouri, and Pennsylvania). Crash data from 1997 through 2006 were obtained for these segments. Results from an EB before-after study indicated that the installation of shoulder rumble strips on rural two-lane highways was associated with a reduction in SVROR crashes by 16 percent. A cross-sectional evaluation indicated installing shoulder rumble strips on rural two-lane highways was associated with a reduction in SVROR crashes by 36 percent. Both results were statistically significant at the 95 percent confidence level and indicated that SVROR crashes tended to decrease when installing shoulder rumble strips on rural two-lane highways. Since the EB method is the preferred to evaluate the safety effectiveness of a treatment, the researchers recommended using that result as a reliable CRF for installing shoulder rumble strips on rural two-lane highways (i.e., 16 percent reduction on SVROR crashes).

Patel and Griffith (2007) estimated the safety effectiveness of installing shoulder rumble strips on rural two-lane highways in Minnesota. Shoulder rumble strips were installed at 24 treatment sites (total length of 183 mi) on rural two-lane highways between 1995 and 2001. SVROR crash data before and after the installation (3 to 9 years in the before period and 3 to 8 years in the after period, depending on the sites) were collected. The researchers analyzed the data using the EB method. Results revealed that SVROR crashes decreased by 13 percent after the treatment was installed. However, this result was not statistically significant at the 90 percent confidence level.

Khan et al. (2015) examined the safety effectiveness of installing shoulder rumble strips in reducing SVROR crashes on rural two-lane highways. Shoulder rumble strips were installed at 38 sites on three highways in Idaho between 2004 and 2007. Roadway segment characteristics (e.g., length, ADT, curvature, shoulder width, etc.) were collected from the state Office of Highway Operation and Safety and Google Earth. The period of crash data collection varied from 2–6 years depending on the year of installation of the treatment at each site. The researchers utilized before-after EB analysis, and the results showed that SVRORs were expected to decrease by 14 percent after installing shoulder rumble strips on rural two-lane highways. This result was statistically significant at the 90 percent confidence level.

Safety and Operational Effectiveness of Edge-Line Rumble Strips

Himes et al. (2017) evaluated the application of edge-line rumble strips on rural two-lane horizontal curves in Kentucky and Ohio. The researchers collected traffic, geometric, and crash data at 229 horizontal curves (15.6 mi) in Kentucky and 579 horizontal curves (42.3 mi) in Ohio and applied the EB method. It was found that crashes were reduced in both states after the installation of edge-line rumble strips. Specifically, CMFs for total, injury, SVROR, nighttime, and nighttime SVROR were 0.79, 0.79, 0.78, 0.75, and 0.71, respectively.

Safety and Operational Effectiveness of Centerline Rumble Strips

Guin et al. (2018) collected 2 years of data on 126 mi of roadways in Georgia where centerline rumble strips were installed in 2005 and 2006. The researchers conducted EB analysis and found that centerline-crossing-associated crashes appeared to decrease by 42 percent after the installation of centerline rumble strips.

Overview of Curve Warning Signs

Often, navigating horizontal curves may cause visibility issues for drivers due to their geometry, roadway configuration, and roadside landscape. To enhance the visibility of a horizontal curve, a variety of signing options are available. Curve warning signs are needed at locations with an advisory speed that is at least 10 mph below the posted speed limit. Similarly, curve warning signs may be appropriate due to geometric features including length, radius, shoulders, or roadside features. In some instances, an unexpected feature may be located within the curve, such as an intersection, geometric change, or something similar. These example characteristics demonstrate the wide variety of issues that ultimately may trigger the need to install static curve warning signs. An enhanced curve warning system can incorporate larger signs, advanced warnings, and in some cases companion flashing beacons to further enhance the curve warning system.

Operational Evaluation of Curve Warning Signs

Tribbett et al. (2000) examined the effectiveness of a dynamic curve warning system that includes elements such as changeable message signs (CMSs), a radar speed-measuring device, cameras, and video detection software. The study evaluated the effectiveness of the CMS and radar unit on driver behavior, showing a reduction in truck crashes.

Gates et al. (2003) assessed different higher-conspicuity materials and their impact on driver behavior and traffic safety. Toward its conclusion, this research used micro-prismatic and fluorescent sign sheeting material to relate it to improving highway safety. In Texas, 14 sites were considered for evaluation, including six curves. Six different fluorescent applications were taken, which included yellow chevrons, yellow chevron posts, curve signs, yellow ramp advisory speed signs, yellow "Stop Ahead" signs, and red stop signs. It was found that fluorescent yellow chevron signs reduced edge-line encroachments as well as curve speed. The other sign treatments led to a reduction in operating speeds. This research found valuable results but was limited and did not examine impacts on the number of crashes.

Although curve warning signs are expected to affect operations, there is evidence of partial adherence by the public to the speeds recommended by these signs. Dixon and Avelar (2011) collected operational speed data at 20 locations in Oregon to assess adherence to advisory speeds by Oregon drivers. The researchers found that 85 percent of the drivers in western Oregon exceeded the advisory speed at horizontal curves by between 8.5 and 19.1 mph and that no less than 61.4 percent of all drivers in that region navigated curves faster than the advisory speed.

Safety Evaluation of Curve Warning Signs

The Accident Investigation Monitoring Analysis (2003) conducted in New Zealand suggests a reduction of 49 percent in crash rates due to the use of chevrons on low-radius curves. Guidelines provided by Austroads (2004) suggest a CRF equal to 15 percent due to chevrons. Chevrons are used over other traffic control devices because they provide better direction and sharpness to the horizontal curve (McGee & Hanscom, 2006; Hallmark et al., 2013). Chevron signs warn drivers of the severity of a curve by delineating the alignment of the road around that curve (Rose & Schoenecker, 2005; IRF, 2006). A study by McGee and Hanscom (2006) reported that the effectiveness of chevron signs in reducing crashes was not established.

Charlton and Pont (2007) studied curve speed management and tested two different groups of curve treatments using a driving simulator. One of the groups was comprised of warning signs to warn drivers to reduce speeds before the curves, and the other group used road markings to affect drivers' lateral displacement. The results indicate that advance warning signs in combination with chevron sight boards and repeater arrows are effective in the reduction of operating speeds. However, this study did not focus on treatments' effects on crashes.

Montella (2009) evaluated the effectiveness of three countermeasures: (a) chevron signs (CMF ranges from 0.41 to 1.92); (b) curve warning signs and chevron signs (CMF ranges from 0.46 to 1.18); and (c) curve warning signs, chevron signs, and sequential flashing beacons along the curve (CMF ranges from 0.23 to 0.62). The countermeasures were measured for 15 curves on the motorway A16 Naples-Canosa, Italy. The characteristics of the curves include small radius, large deflection angle, available sight distance, and super elevation. The results indicated a significant reduction in crashes in different situations, such as daytime, nighttime, non-rainy and rainy, and PDO. The reduction is nighttime crashes (40.8 percent) was higher than daytime crashes (39.3 percent). Similarly, the rainy crashes were higher than non-rainy, and ROR crashes were higher than non-ROR crashes. Another important result highlighted in the study was the dependence of crash reduction on treatment typology. Findings indicated that total crash reduction appeared to be 47.6 percent with the installation of curve warning signs, chevron signs, and sequential flashing.

Srinivasan et al. (2009) analyzed 89 treated curves in Connecticut and 139 treated curves in Washington in terms of geometric, traffic, and crash data to determine the safety effectiveness of curve delineation. Treatments included chevrons, horizontal arrows, and advance warning signs. The results revealed that total injury and fatal crashes were reduced by 18 percent and lane-departure crashes were reduced by 25 percent. Locations with higher traffic volumes and sharper curves showed more significant results than locations with less hazardous roadsides.

Crash Modification Factors for Curve Warning Signs

A cross-sectional study by Galgamuwa and Dissanyake (n.d.) suggested that chevrons result in significant reduction in lane-departure crashes and fatal and injury crashes. The CMF value suggested range was between 0.64 and 0.68, depending on severity level. Similarly, modeling work by Avelar (2012) on the safety effectiveness of advisory speeds modeled safety as a function of curve geometrics as well as the posted speed value, so no straightforward CMF was proposed. However, this research estimated a CRF of 27 percent on average for the conditions represented in the set of study sites in Oregon.

Overview of Lighting at Night for Roadway Safety

Illuminance is the amount of light perceived by the roadway user, which is dependent on the roadway surface and environmental conditions. The strategic positioning of roadway lighting at critical locations, such as intersections or sharp horizontal curves or segments, can help to enhance roadway visibility and therefore reduce nighttime collisions. Transportation agencies often install lighting at locations with a pattern of nighttime roadway departure crashes or most promising sites. In many cases, agencies encounter challenges deploying lighting if electrical service is not available at more remote locations.

Regarding the safety and operational implications of highway lighting, Walker and Roberts (1976) studied the crash data from 3 years at 47 rural intersections before and after lighting was installed. The results showed that the crash rate in the before period was 1.89 crashes per million entering vehicles, whereas the crash rate in the after period was 0.27 crashes per million entering vehicles.

Also regarding the safety and operational implications of highway lighting, another study of Minnesota intersections estimated the relative change in crash frequencies associated with lighting (Preston & Schoenecker, 1999). The results were found statistically significant at the 95 percent confidence level. A reduction of 40 percent in total nighttime crash rates was found. The proportion of fatal and personal injury crashes was found to decrease by 20 percent.

Several previous studies estimated CMF values and calculated safety effectiveness of lighting. A few of the before-after studies indicated that CMF values range from 0.62 to 0.96, and corresponding safety effectiveness is between 4 percent and 38 percent (for example, Harwood et al., 2007).

Also regarding lighting, Green et al. (2003) investigated crash data from nine intersections in Kentucky. The results revealed that there was a 45 percent reduction in nighttime crash frequency after the installation of nighttime lights. A study by Wanvik (2009) evaluated safety effectiveness of lighting at night using crash data from 763,000 injury crashes and 3.3 million property damage crashes. The effect on fatal crashes was higher than the effect on injury crashes. In adverse weather and road surface conditions, road lighting did not show any significant effect. The percentage of crash improvement for pedestrians and bicyclists was higher than for automobile crashes. It was observed that the risk factor on lit roads was 17 percent, whereas on unlit roads, it was 145 percent. The risk factor increased during rainy conditions.

The relationship between road lighting levels and safety was studied by Jackett and Frith (2012) by analyzing crash data and road lighting measurements in New Zealand. The results indicated average luminance to be an important performance measure in predicting crashes on road sections, but the results were not very strong for intersections.

In an HSM validation study by Abdel-Aty et al. (2014) in Florida, the researchers estimated the safety effectiveness of adding lighting at 45 treated and 33 comparison sites, respectively, with similar roadway characteristics and annual average daily traffic (AADT). Results indicated that adding lighting was associated with a reduction in crashes for all crash types and severity levels except for the no-injury crashes.

Gibbons et al. (2015) also studied the relationship between lighting levels and crashes on roadways. The researchers collected crash data from several selected states and the Highway Safety Information System. The results indicated that after around 5 lux, there was no benefit to illumination on an urban interstate. The authors also showed that the lighting requirements could be reduced by 50 percent and still maintain traffic safety conditions.

Overview of Shoulder Widening

Shoulders placed adjacent to travel lanes accomplish several functions, including emergency stop and pull off, recovery area for driver error, and pavement edge support. Shoulder paving is recognized as a positive countermeasure to reduce a shoulder drop-off hazard. *NCHRP Report 633: Impact of Shoulder Width and Median Width on Safety* provides an extensive overview of the safety effectiveness of shoulder widening (Stamatiadis et al., 2009).

Zegeer et al. (1981) analyzed rural two-lane roads in Ohio and Kentucky. The extensive study was conducted with two-lane roadway data collected for 4,950 mi. It was observed that the presence of roadway shoulders inversely correlated with frequency of crash types, such as ROR, opposite-direction sideswipe, and head-on (HO). This study estimated the relation between crashes and multiple roadway geometry features.

Örnek and Drakopoulos (2007) evaluated whether additional unpaved shoulders along with 3 ft of paved shoulders would reduce ROR crash rates. The safety benefits of shoulder widening were evaluated by Gross and Jovanis (2007) in Pennsylvania. A total of 26,000 rural two-lane undivided highway segments between 1997 and 2001 were collected. The two methodologies applied to obtain results were the case-control and cohort methods. The results from the case-control approach indicated that widening shoulders from 2 to 8 ft had an associated CMF of 0.80, while the cohort approach provided a CMF of 0.86. Gross et al. (2009) evaluated the effectiveness of various lane-shoulder widths against roadway departure crashes. A case-control analysis was used for 5 years of geometric, traffic volume, and crash data in Pennsylvania and Washington. The results indicated that a 12-ft lane showed optimal safety when the total paved width ranged from 26 to 32 ft. Lane widths of 11 ft still seemed to be effective when the total paved width was 34 ft.

The CMFs were estimated for fixed roadway lighting and configurations of lane and shoulder widths by studying case-control and cross-sectional methods (Gross & Donell, 2011). The test variables considered were shoulder width and additional shoulder width, and researchers found that at least 4 ft of the unpaved shoulder in addition to existing paved shoulders was associated with a significant safety improvement.

Dixon et al. (2017) conducted a study to determine the shoulder widening needs on the Texas State Highway System. This study developed the criteria for roadway shoulder suitability for pedestrians and bicycles. The researchers analyzed historical crash data at a sample of locations in Texas and found evidence of lower pedestrian/bicyclist injury crashes for widened shoulder roadways.

SYSTEMIC EVALUATIONS OF ROADSIDE SAFETY EFFECTIVENESS

This section of the report summarizes current knowledge about systemic evaluations of roadside safety. The HSIP uses approaches that are mainly based on crash occurrences, usually known as hotspot identification. Under this approach, high-risk sites (defined as sites that experience more crashes than expected) are identified through network screening, and investments are then decided based on the observed crash frequencies (for details, see Hauer et al., 2004).

High-risk sites could be defined as intersections, short segments, or long segments (e.g., sharp curve, narrow lane width). However, this traditional approach may result in recommending more safety improvement projects for urban areas where crashes tend to cluster, rather than rural areas where crashes are more sporadic.

Investment decisions using the traditional approach are based on a site-analysis approach. Such technique focuses on specific locations with a history of severe crashes. According to Preston et al. (2013), the stochastic nature of crash locations over long highway segments makes it more difficult to efficiently predict or estimate the locations where a rarer subset of crashes (say, fatal or severe) would occur on rural highways. As a result, transportation agencies would experience difficulties in meeting safety performance goals by only investing in high-crash locations when traditional techniques are employed. The systemic approach to safety involves identification and implementation of countermeasures that address high-risk roadway factors through system-wide analysis of specific target crash types. Since systemic improvements focus on high-risk roadway features rather than specific locations, it is possible to use the roadway characteristics that are associated with specific crash types to estimate which locations are most likely to experience fatal or severe crashes (FHWA, 2015).

There are many advantages to the systemic safety approach. It addresses specific crash types or crash risk factors, typical countermeasures are low cost and require low maintenance, and it allows agencies to implement a proactive safety program (FHWA, n.d.). When crash or traffic volume data are absent, the use of low-cost countermeasures in systemic projects can help curb the uncertainty of using risk factors for site selection (Gross et al., 2016). Gross et al. (2016) argued that incorporating more systemic projects into a safety program has the potential to address safety problems on a broader scale and enhance the overall effectiveness of a program.

The systemic safety approach is an analytical data-driven process. The intent of the systemic approach is to supplement the traditional site analysis and provide a more comprehensive and proactive approach to prevent the most severe crashes, while the application of the systemic approach yields recommended safety treatments drawing from the roadway system characteristics. The purpose of the systemic safety approach is not to replace the site-by-site analysis; high-crash locations still need to be addressed. However, both the site analysis and systemic approaches provide basis for a comprehensive management program (FHWA, 2015).

FHWA developed a tool for systemic safety project selection based on the current practices for identifying roadway safety problems and developing the HSIP. The FHWA systemic tool provides a step-by-step process for conducting a roadway system safety evaluation. It involves three basic elements: (a) Element 1—the systemic safety planning process; (b) Element 2—a framework for balancing systemic and traditional safety investments; and (c) Element 3—an evaluation of a systemic safety program. The framework of the FHWA systemic tool is shown in Figure 1.

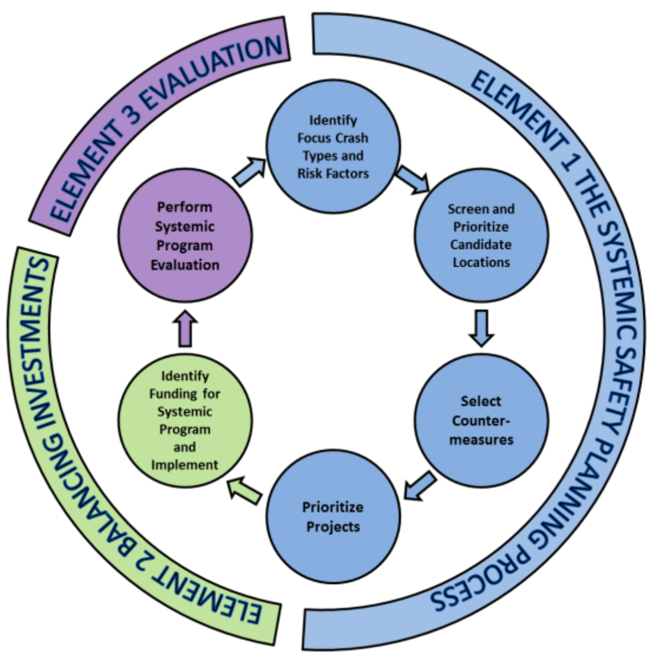


Figure 1. Framework of the FHWA Systemic Tool (Preston et al., 2013).

This research project focused only on Element 1—the systemic safety planning process—and thus it is described in greater detail here. The systemic safety planning process includes four steps: (1) identify target crash types and risk factors, (2) screen and prioritize candidate locations,

(3) select countermeasures, and (4) prioritize projects. The process of the systemic safety planning is shown in Figure 2.

Step 1: Identify Focus Crash Types and Risk Factors

Step 1 is to identify the focus crash types and risk factors that represent the greatest potential to reduce fatalities and severe injuries. Since the focus crash type is ROR crashes in this project, this step refers to the most common characteristics or risk factors for the locations associated with the ROR crashes. The research team considered crash data from 2013 to 2017 in Step 3 and created crash trees to identify the target facilities. A crash tree diagram is an effective tool to illustrate the categorization of crashes to find the target locations or characteristics of interest. The crash tree begins with the total number of ROR crashes in the first level. The crashes are then disaggregated by the area type, road functional class, roadway alignment, posted speed limit, and so forth, until a particular target facility (or characteristic) is found.

Figure 2. Process of Systemic Safety Planning (Preston et al., 2013).

To identify the risk factors, the proportion of ROR crashes for a specific range or value of a variable are then compared to the proportion of existing highway mileage (in case of the roadway segments) within the respective range or value. Table 1 provides the variables that influence the ROR crashes and their data sources that were considered in the risk factor identification in this research.

Table 1. Variables to Be Considered in Risk Factor Identification.

Category	Variables	Source
Roadway Features	Lane width, shoulder surface width/type	TxDOT RHiNo
	Presence of horizontal curvature, curve length, and radius	TxDOT Geo-HiNi
	Presence of shoulder or centerline rumble strips	Google Earth aerial and street views
Roadside Features	Clearance to fixed objects, side-slope rating	Google Earth aerial and street views
Traffic Control Devices	Chevrons, delineators	Google Earth aerial and street views
Traffic Volume	ADT, truck ADT percent	TxDOT RHiNo
Other Features	Posted speed limit, presence of driveways	Google Earth aerial and street views

After the risk factors have been identified, the next step is to evaluate the risk factors in order to rank/prioritize the at-risk locations previously selected based on site and traffic characteristics. In the risk assessment, roadway network elements are prioritized using risk factor weights. Table 2 provides the weights based on the proportion of crash over- and under-representation and crash total when compared to highway mileage supported by Geedipally et al. (2016). The categories "crash over-representation" and "crash under-representation" represent the proportion of observed crashes that are "more than" or "less than" the highway mileage. When crashes at a specific element are over- or under-represented by a certain amount, a specific weight is to be assigned to the element. In addition, weights are to be assigned based on crash total proportion in the specific group of a variable.

Table 2. Risk Factor Weight Criteria.

Category	Weight (points)										
	0	1	2	3	4	5	6	7	8	9	10
Crash Total	≥ 0% and < 10%	≥ 10 and < 20%	≥ 20 and < 30%	≥ 30 and < 40%	≥ 40 and < 50%	≥ 50 and < 60%	≥ 60 and < 70%	≥ 70 and < 80%	≥ 80 and < 90%	≥ 90 and < 100%	100%
Crash Over-Representation	0%	> 0% and < 2%	≥ 2% and < 3%	≥ 3% and < 4%	≥ 4% and < 5%	≥ 5% and < 6%	≥ 6% and < 7%	≥ 7% and < 8%	≥ 8% and < 9%	≥ 9% and < 10%	≥ 10% and ≤ 100%
Crash Under-Representation	0%	> 0% and < 2%	≥ 2% and < 3%	≥ 3% and < 4%	≥ 4% and < 5%	≥ 5% and < 6%	≥ 6% and < 7%	≥ 7% and < 8%	≥ 8% and < 9%	≥ 9% and < 10%	≥ 10% and ≤ 100%

Based on the weights provided in Table 2, the total weight for a particular risk factor can be calculated using Equation 1 (Geedipally et al., 2016).

$$W_t = 10 + CT + CO - CU \tag{1}$$

Where,
W_t = total weight;
CT = weight based on crash total;
CO = weight based on crash over-representation; and
CU = weight based on crash under-representation.

Step 2: Screen and Prioritize Candidate Locations

The second step in the systemic safety planning process is generating a prioritized list of locations. The weighting process is to be applied for all identified risk factors. Lists of prioritized facility elements, such as straight sections and horizontal curves, are to be generated based on the presence of the weighted risk factors—the more risk factors present, the greater chance of occurrence of the ROR crashes and thus the higher probability of being considered as a candidate for safety investments. Once the prioritized lists of facility elements are generated, a visualization map is to be developed using appropriate mapping software such that color coding helps differentiate high-ranked sites from the moderate- and low-ranked sites.

Step 3: Select Countermeasures

The third step in this process is to select highly effective countermeasures to be considered for implementation at candidate locations identified in the above step and to identify/develop the list of high-priority safety improvement projects. Selecting countermeasures is a critical component of the safety management process. The countermeasures anticipated in the process for this project are presented in Table 3. The list is provided for reducing the crash frequency and as well the crash severity. Countermeasures' cost, effectiveness, and timeframe for implementation were adopted from Geedipally et al. (2015).

Table 3. Countermeasures by Crash Types.

Crash Type	Work Code	Treatment	Cost[1]	Effective-ness[2]	Timeframe for Implemen-tation[3]
ROR Crash Frequency	204	Flatten Side Slope	High	High	Short to Medium
ROR Crash Severity	205	Modernize Bridge Rail and Approach Guardrail	Low	High	Short
	206	Improve Guardrails to Design Standards	Low	High	Short
	209	Safety Treat Fixed Objects	Low	High	Short

[1] Cost: low: < \$10,000 per mile or implementation; moderate: \$10,000 to \$100,000; high: > \$100,000.
[2] Effectiveness: low: CMF > 0.9; moderate: 0.7 < CMF ≤ 0.9; high: CMF ≤ 0.7.
[3] Implementation: short: less than a year; medium: 1 to 2 years; long: longer than 2 years.

Step 4: Prioritize Projects

The next step in this process is to conduct a B/C analysis for the top locations identified in the previous step with the expected crash frequency and severity reductions, implementation, and maintenance costs during the service life. This step uses the Safety Improvement Index (SII) for the B/C analysis to prioritize safety projects and the cost-effective countermeasures. This B/C analysis using the SII can be applied for all the network locations identified and for candidate countermeasures. Selection of countermeasures for deployment is to be determined based on best safety improvement and lowest cost to implement. This effort should yield recommended countermeasures that are feasible by cost, effectiveness, and implementation timeframe.

The decision-making process that includes a set of criteria such as volume, environment, adjacent land use, or cross-section is to be used to identify the appropriate countermeasure for high-priority locations. The decision-making process in the systemic approach does not just identify the most appropriate countermeasure for each individual location, as done when addressing hot spots, but also considers multiple locations with similar risk characteristics, selecting a preferred countermeasure or countermeasures that are appropriate and affordable for widespread implementation. To assist with this process, a decision tree of countermeasure selection depending on network elements of the projects should be developed.

OVERVIEW OF SYSTEMIC SAFETY TOOL IMPLEMENTATION BY VARIOUS STATE DOTS

FHWA has compiled several case studies of systemic approaches used by different states. For example, Minnesota has taken a systemic approach through the development of safety plans (FHWA, 2012). Following the procedures in FHWA’s systemic approach to safety, the Minnesota Department of Transportation (MnDOT) first identified the crash types and risk factors. MnDOT analyzed severe crashes (fatal [K] and incapacitating injury [A] crashes) on all public roads. The focus crash types were lane-departure crashes and intersection-related collisions. After identifying potential risk factors for facilities, MnDOT evaluated each of the factors using descriptive statistics. The risk factors were reported for curves, intersections, and segments separately.

The Illinois Department of Transportation developed a document titled *Systemic Safety Improvements: Analysis, Guidelines and Procedures* that is similar to FHWA’s Systemic Tool (CH2M Hill 2011). The guidelines can be used for identifying high-priority areas to integrate safety into projects and plans throughout the transportation management process. The document details the systemic process, which includes collecting data, organizing the data, obtaining critical values, compiling the results, conducting a field assessment, and selecting countermeasures.

The Louisiana Local Technical Assistance Program (LTAP), which administers the Local Road Safety Program, has been working to implement a systemic approach to improving safety on horizontal curves (FHWA, n.d.). The Louisiana LTAP has developed a process to characterize and prioritize curves based on certain criteria and to develop a manageable process to implement projects systemically.

MoDOT noted the need to apply a systemic approach to reduce fatal and serious injury crashes. MoDOT invested HSIP funding into resurfacing projects on major roadways, where nearly half of all fatal crashes occurred (FHWA, n.d.). The concepts and procedures MoDOT adopted were similar to those used by MnDOT. However, MoDOT evaluated the effects of edge lines that were installed on 570 mi of rural two-lane state highways. The evaluation results indicated that installing edge-line markings led to a 15 percent reduction in total expected crashes and a 19 percent reduction in severe expected crashes. The results were used by decision-makers to determine whether or not to continue funding as normal or implement a particular countermeasure that focused on crash types on specific facilities. The findings strongly suggest that limited safety funding could be appropriately directed to projects and locations that produce safety benefits in terms of reduced crashes for the least investment (Storm et al., 2013).

The Nebraska Department of Roads (NDOR) conducts a systemic safety analysis to identify potential horizontal curves for safety improvements based on risk (FHWA, n.d.). Mainly, NDOR uses the systemic approach for its county sign installation program.

The Ohio Department of Transportation (ODOT) Office of Local Programs administers a systemic signage intersection and curve upgrade program for targeted Ohio Townships (FHWA, n.d.). The Ohio LTAP Center provides crash data and information on the types of sign packages available for specific situations. Townships can choose from the signage packages or build their own sign orders. The ODOT Office of Local Programs also provides guidance and assistance to townships on sign installation if necessary.

The Kentucky Transportation Cabinet (KYTC) applied a systemic approach in five of the state's counties (FHWA, n.d.). The application was based on the previously conducted systemic planning, which focused on roadway departure crashes on the state's highway system. KYTC identified and considered five potential risk factors: (a) horizontal curve density, (b) lane width, (c) shoulder type, (d) shoulder width, and (e) speed limit. Each risk factor was associated with a threshold value. Analysis indicated that the curve density and shoulder types were generally the determining factors for high-risk scores (FHWA, n.d.). As a result of its analysis, KYTC implemented a set of cost-effective countermeasures on curves. Although effects of these countermeasures have not been evaluated, the systemic approach has been shown to be an easy-to-apply process to evaluate roadways in Kentucky. In addition, the systemic analysis conducted by KYTC was entirely based on available photo logs, so it did not require extra work for gathering additional data (FHWA, n.d.).

The New York State Department of Transportation (NYSDOT) used a systemic approach to identify sites where high-risk crashes could be reduced by implementing low-cost roadway countermeasures (FHWA, n.d.). NYSDOT started the systemic planning by analyzing crash data. The data suggested that road-departure and intersection-related collisions were the two primary types of crashes statewide. NYSDOT selected lane-departure crashes as the focus crash type. The analysis suggested that the most serious lane-departure crashes occurred on two-lane, rural state highways with a posted speed limit of 55 mph. NYSDOT compared the severity of crashes at locations with similar risk factors and discovered that three characteristics were over-represented: (a) AADT between 3,000 and 5,999, (b) curve radii between 100 and 300 ft, and (c) shoulder width between 1 and 3 ft.

The Public Works Department of Thurston County, Washington, used a systemic approach to explore the potential benefits of proactive safety planning. The Public Works Department selected roadway departures in horizontal curves as the focus crash type because the assessment of crash data suggested that 81 percent of severe curve crashes occurred on arterial and collector roadways within the county (FHWA, n.d.). The department identified nine risk factors from a list of 19 potential factors. Each factor was given an ordinal score based on the level of confidence. The evaluators then calculated the number of risk factors present for each of the segmented roadway curves. The department identified four low-cost, low-maintenance countermeasures that were systematically implemented at the curves. These included (a) traffic signs (chevron and large arrow signs), (b) pavement markings, (c) shoulder rumble strips, and (d) roadside improvements (object removal, guardrail, and slope flattening) (FHWA, n.d.).

In Texas, Walden et al. (2015) developed a methodology for identifying, evaluating, and prioritizing systemic improvements. The authors mostly followed the steps proposed in the FHWA tool, except that they noticed that the higher-volume roads always tended to have more crashes. To remove the biased selection of higher-volume roads, the researchers divided the highways into three categories: low volume (<400 vpd), moderate volume (400 to 1,200 vpd), and high volume (>1,200 vpd). TxDOT used this methodology to develop a systemic process for selecting the roads for highway widening (Geedipally et al., 2015). The primary risk factors identified were lane width, truck volume, shoulder width, and presence of sharp horizontal curves. The study found that the horizontal curves with less than 1,000 ft radius are common risk factors for SVROR and HO KA crashes on rural two-lane highways in all traffic volume groups (Geedipally et al., 2015). The second application of a systemic approach in Texas was the selection of projects for median barrier installation (Geedipally et al., 2016). The primary risk factors that were identified include median width (unprotected median), inside shoulders, and truck percentage. The next application developed by TxDOT was a systemic approach to project selection for improving horizontal curve safety (Geedipally et al., 2016). The authors used the horizontal curve data and identified risk factors that included lane and shoulder width, truck proportion, curve radius, and deflection angle. This study presented a list of candidate countermeasures with their effectiveness, cost, and service life.

CONCLUSIONS

This chapter summarized the results from Task 2 of TxDOT Research Project 0-6991. The chapter synthesized the findings of a comprehensive literature review of commonly used safety countermeasures for roadway departure crashes, providing an overview of each countermeasure followed by results of studies that quantified the countermeasure's estimated safety, operational, and economic effectiveness. This chapter also presented an overview of the systemic approach to the application of safety countermeasures, current practice by other DOTs, and recent applications of the approach in Texas. The following subsections synthesize the key findings of this effort.

Safety Countermeasures

This research found a few studies providing evidence of the safety effectiveness of roadside flattening to prevent either SVROR or ROR crashes. Safety effectiveness as a CRF has been estimated to range between 2 percent and 35 percent depending on the study and crash type.

However, roadside flattening is costly, and it could be cost prohibitive to apply in a systemic way.

Regarding the safety effectiveness of guardrails, evidence is clear about the associated reduction on crash severity (ranging from 45 percent to 92 percent, depending on the study location, crash type, and whether the treatment was applied on the roadside or on the median). Evidence is more diffuse about the safety effectiveness in terms of crash frequency. While some studies have found slight increases in crashes when guardrails are present, some studies have found the opposite. Various researchers have offered their explanations as to why an increase or a decrease in crash frequency may be expected.

Regarding median-specific crash attenuators, the evidence in the literature is scarce but rather suggestive of a safety benefit. Many studies evaluated this countermeasure in combination with roadside attenuators (i.e., guardrails), though some studies looked specifically to the effectiveness of the cable barrier applied in the median.

HFST has been studied more recently and found effective against wet-weather crashes, severe crashes, or a combination of these crash types by multiple researchers. The safety effectiveness has been estimated within the range of 25 percent to 75 percent.

Studies on the safety effectiveness of pavement markings have suggested safety benefits ranging from 17 percent to 37 percent in crash reductions, depending on the study-specific conditions and crash types.

Studies on the effectiveness of rumble strips suggest safety benefits ranging from 16 percent to 79 percent. Most studies place the size of the CRF for shoulders between 10 percent and 20 percent. Similarly, the CRF of edge-line rumble strips have been estimated between 21 percent and 29 percent.

Regarding curve warning signs, various studies have suggested crash reductions ranging between 25 percent and 40 percent.

Earlier estimates of the safety effectiveness of lighting have suggested up to 40 percent reduction in crashes. More recent work has suggested more moderate safety effectiveness, or even no effectiveness from high-granularity data when accounting for time series and operational conditions at ramps and intersections (Gibbons et al., 2015).

The evidence of the effectiveness of shoulder width is ample in the literature, either by estimated safety shifts (up to 14 percent CRF) or by its appearance among risk factors in past applications of the systemic approach to the application of roadside safety countermeasures.

Systemic Approach

Every year, rural highways in Texas experience a considerable number of fatal and severe crashes because of vehicles leaving the traveled way. Because these crashes are not evenly distributed across the many miles of rural roadways, it is often difficult to isolate high-crash locations for safety improvements. Thus, the traditional safety methods make it more difficult to efficiently predict or estimate the locations to implement countermeasures. The systemic approach to safety, which is a proactive approach, involves improvements that are widely implemented based on high-risk roadway features correlated with the roadway departure crashes.

Many states have started using the systemic approach to address severe crashes. Texas used the systemic approach for various applications, such as highway widening, median barrier installation, and horizontal curve safety improvement. The research team adopted the methods developed in Texas and adapted an approach for the installation of roadside treatments. The variables that were considered were categorized into five groups, as presented in Table 1, and are discussed in more detail in later chapters of this report.

Impact of Findings to the Project

The findings of this review of literature show that, in general, past works have supported the notion that roadside countermeasures have measurable safety effectiveness. Additionally, the systemic approach has been successfully implemented in other states as well as in Texas for applications other than the roadside condition.

Informed by these findings, the research team prepared a list of countermeasures to be further studied in this project, as described in the next chapter of this report.

CHAPTER 3. DATA COLLECTION

This chapter summarizes the data collection activities undertaken to support this research. Two parallel data collection efforts are documented: (a) data collection to support the application of a systemic analysis, as described in the prior chapter; and (b) data collection to revise and update work codes for select roadway departure crash countermeasures.

The chapter is structured in four parts: (1) preliminary definitions; (2) data collection activities in support of a systemic analysis; (3) data collection activities in support of updating roadway departure work codes; and (4) chapter summary.

PRELIMINARY DEFINITIONS

Prior to describing the data collection activities, the research team studied different definitions of roadway departure crashes and adopted a strategy to define crashes of interest for this study. According to a memorandum from FHWA (2014), the new definition of roadway departure (RwD) crash is the following:

> A crash in which a vehicle crosses an edge line, a center line, or leaves the traveled way is a roadway departure crash. The vast majority of RwD events are captured in FARS by finding crashes in which the first event for any vehicle involved in the crash is one of the following: (63) Ran Off Road – Right, (64) Ran Off Road – Left, (65) Cross Median, or (68) Cross Center Line. In addition, there are a number of fixed-object codes included based on the idea that a vehicle must have left the roadway in order to impact that object as a first event. Those fixed-object codes include 17, 19-43, 46, 52, 53, 57, and 59. Lastly, three other event codes were deemed to most likely be indicative of a roadway departure, those being (67) Vehicle Went Airborne, (69) Re-entering Roadway, and (71) End Departure. The single change to the coding because of the 2014 updates was to remove the intersection filter (i.e., roadway departure crashes include non-intersection and intersection locations).

In many cases, the police reported that crash data did not provide extensive reporting of sequence of events for all reported crashes. Lord et al. (2011) explored different definitions of RwD crashes in the TxDOT report *Analysis of Roadway Departure Crashes on Two-Lane Rural Roads in Texas*. After examining different definitions, the study reported, "The definition D1 (Collision ≤ 5 and Road_relat = 2, 3, or 4) was the most inclusive definition, identifying at least as many crashes as any other definition studied." The research team found that the Texas Strategic Highway Safety Plan (SHSP) (2017–2022) also used definition D1 to label ROR crashes (TxDOT, TTI 2017). The research team also used the same definition (see Table 4) in this study.

Table 4. Definition of ROR Crash.

Crash Types and Location	Definition	CRIS Data Codes
Run-Off-Road Crash—All	A single-vehicle crash where the impact of the first harmful event occurred on the shoulder, beyond the shoulder, or in the median of the roadway.	ROAD_RELAT_ID VALUES = 2 = Off Roadway, or 3 = Shoulder, or 4 = Median, and COLLSN_ID = 1 = OMV Vehicle Going Straight, or 2 = OMV Vehicle Turning Right, or 3 = OMV Vehicle Turning Left, or 4 = OMV Vehicle Backing, or 5 = OMV Other

Detailed data were necessary in order to achieve the objectives of this project. Specifically, the two analysis tasks of this project consisted of (a) a systemic analysis of departure crashes, documented in Chapter 4; and (b) development of updated work codes for roadside countermeasures, documented in Chapter 5. Considering these two specific tasks, their objectives, and their data needs, the data collection activities were naturally divided into two main efforts, each supporting a specific subsequent task. The two following subsections summarize the data collection activities for each of the two evaluation efforts just described.

DATA COLLECTION FOR SYSTEMIC ANALYSIS

In order to perform a systemic analysis of roadway departure crashes in Texas, the research team searched key variables in current TxDOT databases for availability and completeness.

Upon completion of this review, the research team determined that manual data collection would be required to develop a database that would support a systemic analysis. Table 5 shows the databases and sources that were identified for data collection.

Table 5. Databases and Sources Identified for Data Collection.

Databases	Source	Purpose
CRIS	TxDOT	Crash data
GRID	TxDOT	Roadway and traffic
RHiNO	TxDOT	Roadway and traffic data, sampling frame
Street View	Google Imagery	Roadway characteristics
Satellite Photographs	Google Imagery	Roadway characteristics

Because manual data collection was deemed necessary, the first challenge to this effort was to determine the amount of data that were feasible to collect. The following subsection summarizes the steps the research team followed toward making said determination.

Probability Sample of Two-Lane Rural Roads in Texas

Because statewide data collection was unrealistic, considering the timeframe and resources available for this project, the research team decided to develop a probability sample of two-lane roads in Texas for detailed data collection. The rationale of this step was that Texas-wide representativeness of the results could be achieved if the data were collected from a truly representative sample of roads. Probability sampling is a set of principles and methodologies to systematically select a sample in such a way that it is possible to quantify the uncertainty present in the sample estimates with respect to the quantities of interest in the complete population. The key of probability sampling is that any one datum in a population has a finite, no-zero probability of being selected into the sample prior to data collection. Based on this feature, any sample estimate can be characterized in terms of how much it represents the estimand (i.e., quantity being estimated) at the population level.

Sample Design and Target Precision

The research team developed a probability sample of the state two-lane roadways from a probability sample that would allow researchers to draw inferences about quantities of interest at the sampled population level (the population in this case being all miles of two-lane highways in Texas maintained by TxDOT). The sampling frame for probability sample design can be controlled effectively using key variables available from the Road-Highway Inventory Network (RHiNO). The research team proposed using a stratified sample balanced at key variables. The stratification criterion was TxDOT's four regions (north, west, south, and east). The balancing variables were selected from RHiNO such that they are known to be associated with the safety performance highways, namely:

- AADT.
- Truck AADT.
- Lane Width.
- Shoulder Width.
- Section Length.

The method selected to draw the equal-probabilities sample was an implementation of the fast algorithm proposed by Chauvelt and Tillé (2006), based on cube sampling methods. More details on this procedure can be found elsewhere.

Sample Size Determination by Resampling

The sample size was determined via resampling procedures on the sampling frame. The cube sampling procedure was repeated 100 times with replacement for various potential sample sizes. From this procedure, the researchers estimated the design precision of key estimates that would be obtained from applying a safety systemic analysis at each sample drawn. A minimal target precision was determined at 10 percent. In other words, the proposed sample should be of a size that yields estimates that would be within 10 percent of the statewide value at the most. The following plots represent the results for a sample size of n = 600 roadway segments.

Figure 3 shows the results of the sample estimate of total target crashes from 100 resampling iterations for a 600 sample size. Since this total is known from the sampling frame (i.e., an arbitrary definition of crashes in the RHiNO file, set at 50 percent of all RwD crashes for the resampling exercise and shown as a blue line in the plot), two errors can be estimated: the expected error directly obtained from the sample design, and the resample error from the direct comparison between the parameter (blue line) and each iteration. It can be seen that the resample error is smaller than the expected error, but in the same order of magnitude as expected.

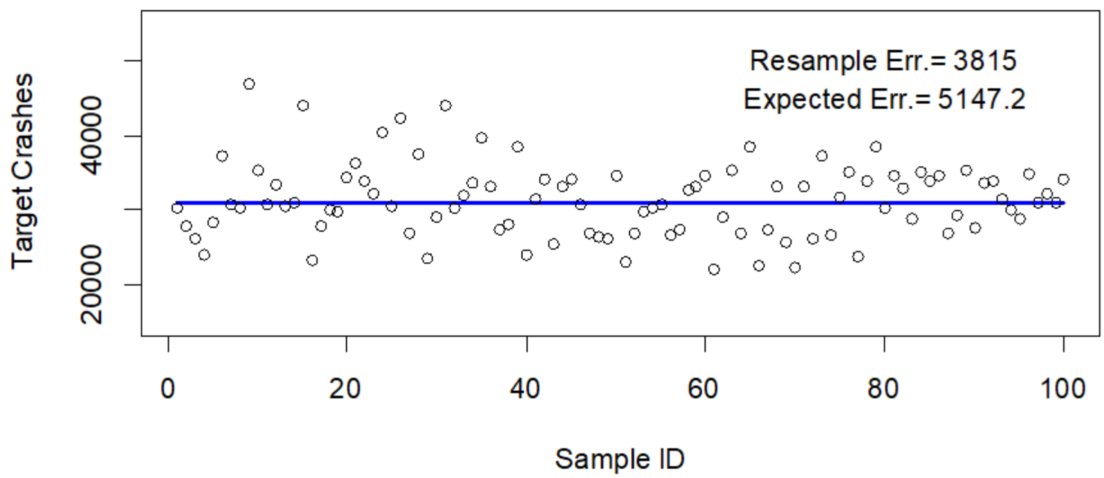


Figure 3. Resampling Results for Sample Target Crash Estimate (n = 600).

Figure 4 shows the results from estimating the proportion of target crashes. In this case, the closed form of the expected error is not known nor trivial to obtain. However, the resampling error can be as easily obtained as before, which was found to be 0.06, or 6 percent. This is an acceptable level of precision since it is smaller than 10 percent.

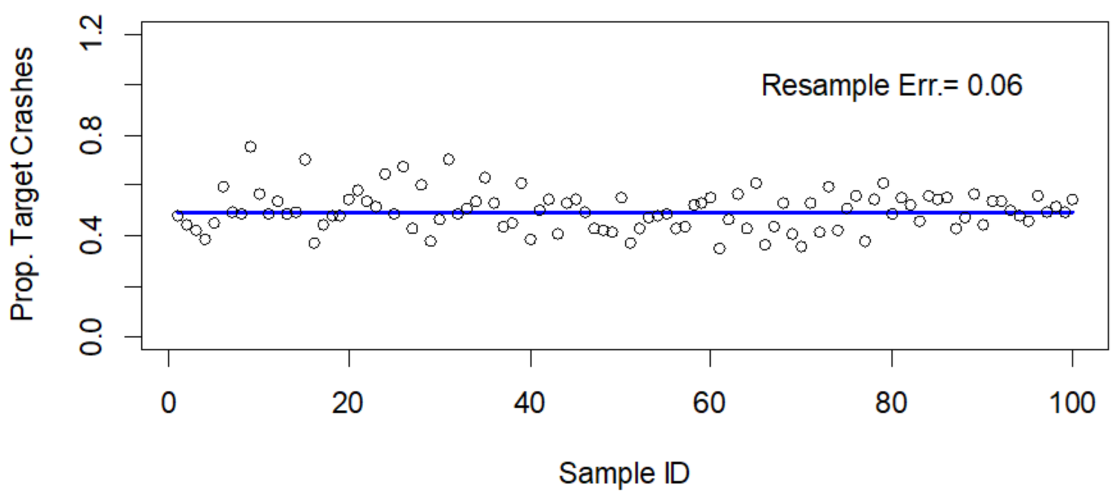


Figure 4. Resampling Results for Proportion of Target Crashes Estimate (n = 600).

Similarly, Figure 5 shows the resampling results for estimating the statewide over-representation of target crashes using only the segments in the sample. The resampling error was found to be 0.05, or 5 percent. This is an acceptable level of precision since it is smaller than 10 percent.

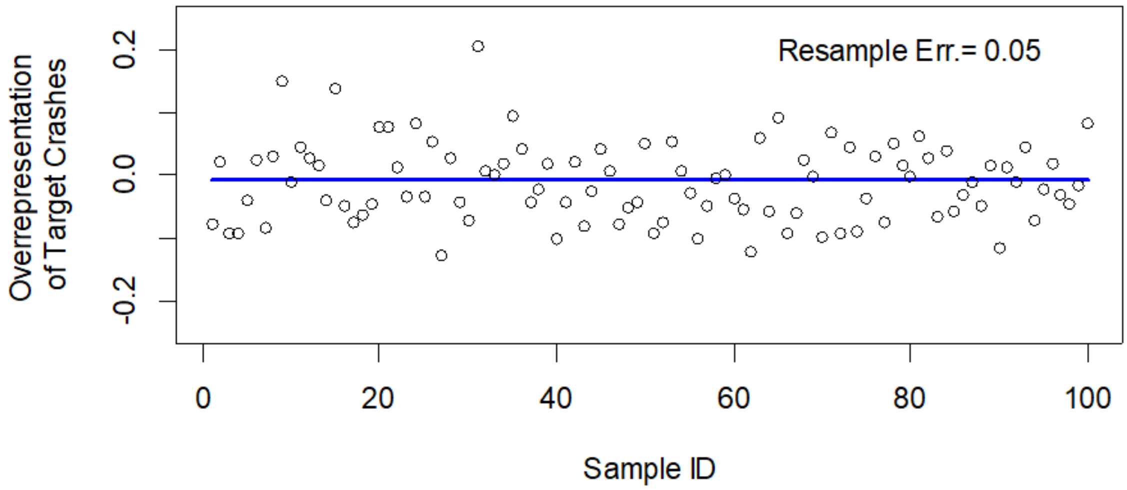


Figure 5. Resampling Results for Statewide Over-Representation of Target Crashes Estimate (n = 600).

Given the results from this exercise, the research team determined that an initial sample of 600 randomly selected highway segments would produce results with slightly better accuracy than desired. Because it is inevitable to lose some data points as the data collection progresses, the research team anticipated that the final accuracy in the data sampled would move closer to the target of 10 percent after cleaning the sample database.

Data Collection from Probability Sample

As mentioned earlier, the research team identified a total of 600 segments across the state of Texas for data collection (the cumulative road length of these segments is 353 mi). Figure 6 shows the geolocations of all the 600 segments in the probability sample. It can be noted that this sample adequately covers the geography of the state of Texas. The resulting geographic dispersion obtained from a balanced stratified sample is expectedly representative of the state, but it may pose a significant challenge if manual (i.e., on-site) data collection were to be collected. Fortunately, all data collection required tools and methods that spared the research team from the burden of on-site visits. Therefore, it was feasible for a team of student workers to assist with the data collection activities.

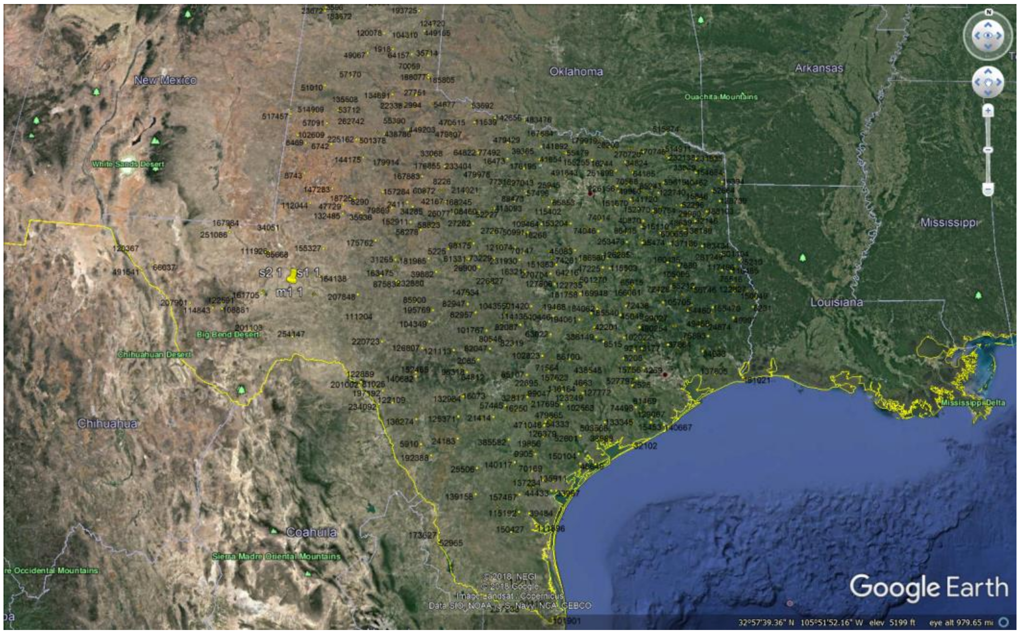


Figure 6. Probability Sample Roadway Segments.

As an initial step, two KMZ files were created showing the geolocations of the selected sites. The data collection process involved collecting the roadway design characteristics, such as lane width, clear zone width, horizontal curves, and so forth. The data collection process was divided into four phases. To facilitate the data collection process in each incremental phase would require the use of the following software packages: Google Earth Pro (GE) (Google Inc. 2019), Google Maps Street View, R Statistical Software (The R Development Core Team 2013), and MS Excel.

In the first phase, data were collected from GE Aerial View. The second phase consisted of marking all the curves in the segments by using markers for later calculation of curve radii and other curve parameters. This was done using Google Earth Pro. The third phase of data collection involved Google Street View to capture details of on-road characteristics not clearly visible from GE Aerial View. Finally, in the fourth phase, static images were downloaded from Google Street View to be analyzed using a package for image analysis in R Statistical Software. All data collected were recorded in Excel spreadsheets. After a quality-control (QC) phase, all spreadsheets were merged into a final database for analysis. The data collection protocols are provided as appendices to this report.

Phase I: Aerial View Data Collection

Roadway geometric design characteristics that could be measured from the aerial view in Google Earth Pro were measured in the first phase of the data collection process. The roadway characteristics obtained in this stage are shown in Table 6.

Table 6. Variables Obtained in Phase I Data Collection.

VARIABLE	UNITS
Length of the Segment	Miles
Revised Length	Miles
Lane Width	feet
Number of Horizontal Curves	Count
Left Paved Shoulder Width	Feet
Right Paved Shoulder Width	Feet
Full Guardrail Length (both sides)	Feet
Guardrail Distance from Paved Shoulder	Feet
Clear Zone Width	Feet
Number of Driveways	Count
Number of Minor Intersections	Count

The Ruler tool in Google Earth Pro was used to capture linear measurements in Table 6. The following are some of the features identified during the data collection process in Phase 1:

- The segments have uniform characteristics throughout the segment, but to get more accurate values, multiple measurements for the same segment were recorded to calculate the average lane and shoulder widths of the respective segments.
- The length of the segments was found out to be present in the RHiNO database. After confirming the data with a few actual measurements (calculated from Google Earth Pro), it was decided that the length of the segment would be directly extracted from RHiNO and not measured every time.
- For segments starting or ending at a major intersection, segment length was revised to avoid the sphere of influence of an intersection. The revised segment length is equal to actual segment length—intersection sphere of influence was assumed between 250 ft and 400 ft.
- All data were recorded in an Excel spreadsheet template created prior to beginning the data collection.

Figure 7 shows an example lane width calculation using Google Earth Pro. The detailed data collection protocol is shown in Appendix I.

Phase II: Horizontal Curves Data Collection

For the second phase, the research team identified horizontal curves in each segment. The curves on each segment were marked for the calculation of curve radii, chord length, and other horizontal curve calculations. Google Earth Pro was used to place the pins along the alignment.

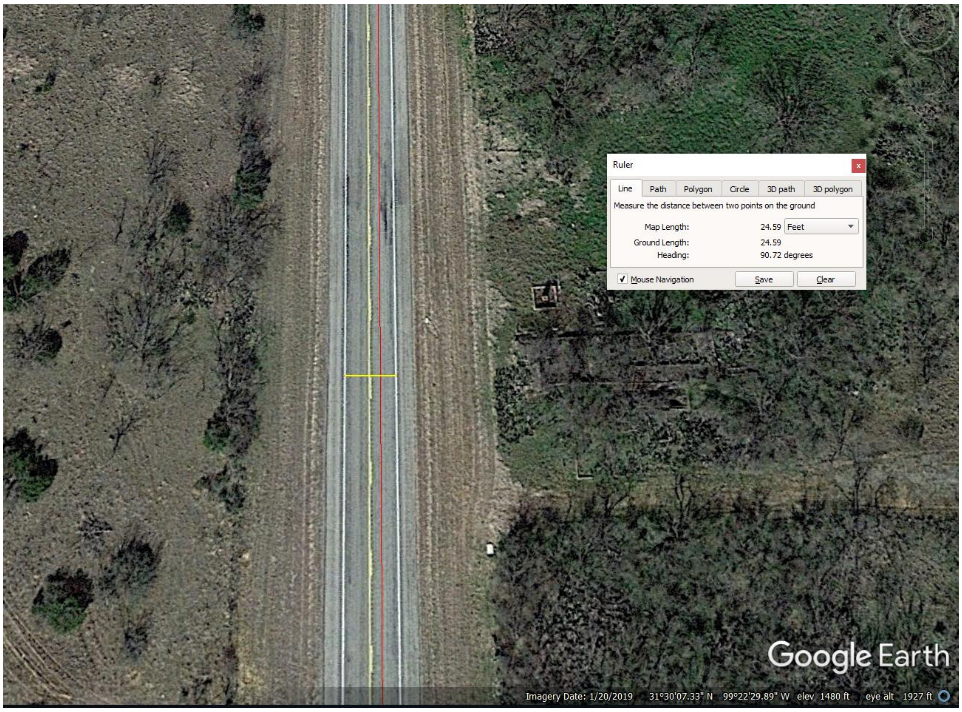


Figure 7. Lane Width Calculation Using Ruler Tool in Google Earth Pro.

The files with all key alignment data were then used to estimate curve data: length, radius, and deflection angle. The detailed data collection protocol is attached in Appendix II.

Some highlights of the Phase II data collection process are shown next.

- Each curve was marked with a minimum of seven markers along the alignment to ensure accurate estimation of curve data.
- Curves with a horizontal length of < 200 ft between point of curvature and point of tangency were considered as flat curves and were excluded from further processing to estimate curve data.
- All the markers on the curve in each segment were saved in one KMZ file in a folder that was then exported to a tool that would estimate the curve data.

Figure 8 shows a sample site with markers as described above. All marker data were exported into a spreadsheet equipped with macros to expedite the processing. The total number of horizontal curves identified and processed in this phase was 424.

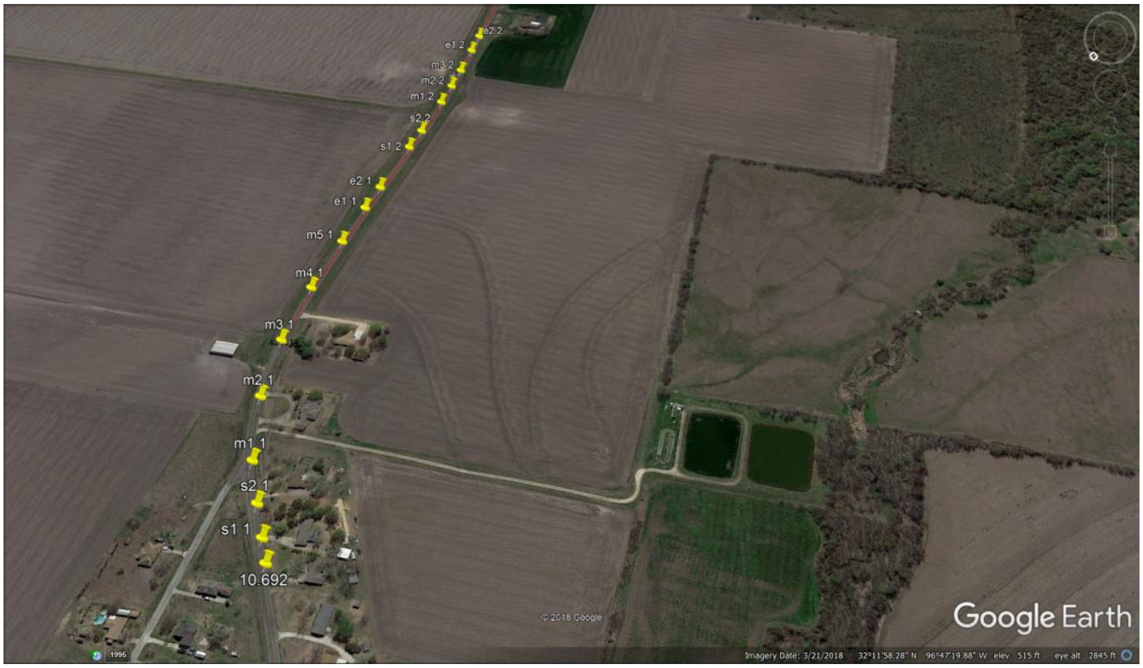


Figure 8. Sample Site with Horizontal Curve Markers.

Phase III: Google Maps Street View Visual Analysis

The third phase in the data collection process involved checking for the presence (or absence) of certain physical characteristics of the roadway segment. This process was a systematic review of the roadway segments using Google Maps Street View to collect the data. The segments were studied for the features listed in Table 7.

Table 7. Features Obtained in Phase III Data Collection.

FEATURE	DESCRIPTION
Problem Flag	Variable indicative of an issue with the site or metric
Shoulder Rumble	Presence or not
Center Rumble	Presence or not
Speed Limit (mph)	Actual value observed in image
Chevrons	Presence or not
Delineators	Presence or not
End Terminal Type	Actual type observed in image
Number of Poles (both sides)	Presence or not
Number of Lone Trees	Presence or not
Number of Cluster of Trees	Count

The detailed data collection protocol of these procedures is documented along with Phase I procedures in Appendix I. Some of the key highlights of Phase III data collection are listed next.

- The presence or absence of certain roadway characteristics was recorded by coding indicator variables, taking the value of 1 if the feature was present and 0 otherwise.
- If the two trees were in proximity of each other (<50 ft), they were considered a cluster of trees as opposed to two lone trees.
- Segments that had undergone construction between 2011 and 2017 were flagged. Similarly, segments with poor photo quality, more than three curves, and discrepancies in street view and aerial view were also flagged.
- These data were collected in an Excel workbook to be merged with the rest of the data collected in other phases.

Figure 9 shows a typical image used in Phase III that indicates the presence of median and edge rumble strips, as well as the end terminal type for the guardrail.

Figure 9. Sample Image from Phase III.

Phase IV: Data Collection for Image Analysis

The last phase in the data collection process was analyzing images of segment cross-sections to measure features that could not be measured in aerial view, such as guardrail height, width of the utility poles, side slopes, and so forth. This process consisted of two parts: (1) estimating a set of calibration parameters (spatial orientation of the camera, field of view, lens focal length, and camera sensor geometry); and (2) collecting Google Street View images to be processed in R Statistical Software.

Calibration

The purpose of the calibration phase was to estimate the parameters needed to perform image analyses using calibration images with known distances. Initial calibration was attempted by measuring all available horizontal distances only (all visible lane widths, paved shoulder width, etc.). However, it was determined that this procedure would not yield reliable estimates; some estimates had significant margins of error. Therefore, the researchers performed tests using different areas of calibration. Different URL parameters corresponding to field of view (FOV) and tilt on Google Street View images were tested to determine highest accuracy.

After testing various URL parameters for multiple sites, a few valid calibration results were determined. The researchers conducted further measurement tests using those results and determined that they produced similarly reasonable values of other measurements (different than the calibration values). Figure 10 shows a sample calibration image.

Figure 10. Sample Calibration Image.

Since the bounded algorithm takes in an upper as well as a lower bound, the various potential calibration results were used to determine a more accurate range of values to feed the algorithm in the second part of this phase.

When running the tests, two competing optimization algorithms were considered—the L-BFGS-B and Nelder-Mead—with different degrees of success and resulting accuracy. Because of its reduced amount of error and efficiency, the researchers selected the Nelder-Mead algorithm to ultimately estimate the calibration parameters.

Calibration Results

Multiple combinations of FOV and camera tilt were tested from street view image parameters. The average error estimated in each calibration ranged from as little as 0.07 ft up to 0.88 ft. The calibration parameters selected were those with average estimated errors of 0.25 ft or less. Further street view images from east, west, south, and north regions of Texas were tested to verify the calibration results.

It was found that Google Street View images with a tilt of 85 degrees and FOV of 60 returned the most acceptable results, with errors within acceptable limits of 0.25 ft.

Downloading Images from Google Street View for Image Analysis

For each segment, images had to be collected for analysis in R Statistical Software using the calibrated values as determined in the section above. Using Google Street View, multiple images were downloaded for each segment. The detailed protocol for downloading the images is described in Appendix III. The downloaded images were prepared for analysis and analyzed using R Statistical Software, as described in the next section.

Image Analysis in R Statistical Software

Images downloaded from Google Street View for each segment were organized into a database for QC checks prior to being analyzed in R Statistical Software.

Table 8 shows the dimensions that were measured in Phase IV data collection. Distance of electric poles, lone trees, and cluster of trees from the sides of the shoulder were measured. Also, for segments having guardrails, the guardrail height and the height and width of the end terminals were also estimated using R Statistical Software. The maximum side slopes from street view images were calculated, and the uniformity of the side slopes on both the sides of the road as well as at different points of cross-sections were checked at segments picked at random for QC. Also, all segments having variable speed limits were identified.

Table 8. Features Obtained in Phase IV Data Collection.

FEATURE	DESCRIPTION
Pole Distance (ft)	Distance from pavement edge to pole
Guardrail Height (ft)	Height from paved surface
End Terminal Width (ft)	Width edge to edge
End Terminal Height (ft)	Height from paved surface
Lone Tree Distance (ft)	Distance from pavement edge to tree
Distance to Cluster (ft)	Distance from pavement edge to closest tree
Maximum Side Slope (ft)	Estimated from edge of shoulder
Uniformity of Side Slopes	Qualifier variable based on data analyst judgement
Variable Speed Limits Segments	Indicates if more than one Speed limit value is available for the segment
Position of the Shortest Curve	Indicates if the shortest curve is either on an extreme (if multiple curves are present) or not

Since measuring these quantities was not feasible with any other tools used in the previous phases, it was key to validate any metrics that could be corroborated through another independent measurement. The research team performed this validation effort by measuring lane and shoulder widths from the street view images because those values could be compared to the values obtained in Phase I. It was confirmed that the differences were acceptable in most cases, and recalibration was found necessary in a few cases where significant discrepancies between the values were found.

Clear Zone and Roadside Objects

During data collection, the number of lone trees, poles, and clusters of trees within 50 ft from the edge of the lane were identified. The length of any three clusters of trees was measured from the aerial view, and the number of trees present in each cluster were counted from the street view perspective. The actual number of trees were counted if the cluster length was found to be less than 200 ft. With this information, the average length of each cluster of trees and the average number of trees in each segment could be calculated. However, since the roadside condition varied along segments, in general, the clear zone of each segment of analysis was not clearly defined using only the lateral offsets and dimensions of the roadside objects. The next subsection describes further steps used to define the clear zone with an additional function proposed by the research team: the equivalent length of influence of roadside objects.

One key element to be defined is the bounds of the influence of a roadside object on the edge of the road. For example, even if a tree is only 3 ft wide, there must be a small length of road before the tree from which it is most likely that a departing vehicle would hit the tree. Such length of the road (called equivalent length of influence from this point forward) is longer than 3 ft in all likelihood. In order to estimate the equivalent length of influence along the road corresponding to a given object on the roadside, the research team determined a range for the typical departing angle of a vehicle, as well as the lateral offset of roadside objects and clusters of trees along the road. When jointly considering equivalent length of influences of all roadside objects, an average clearance zone for each segment can be determined as a weighted average of all lateral offsets using their equivalent lengths of influence as weights. In the context of this research, clear zone is defined as the distance from the edge of the travel lane to the roadside object, which can be a tree, cluster of trees, pole, and so forth. The research team reviewed relevant literature on departing angle to better inform the range to be used in the calculations just described. The results from that review are shown in the next subsection.

Literature on Departure Angle

The angle of departure, also known as the encroachment angle, is the angle at which the vehicle departs from the roadway in case of ROR incident. It is normally measured by considering the edge line of the pavement and the direction of the skid marks (see Figure 11).

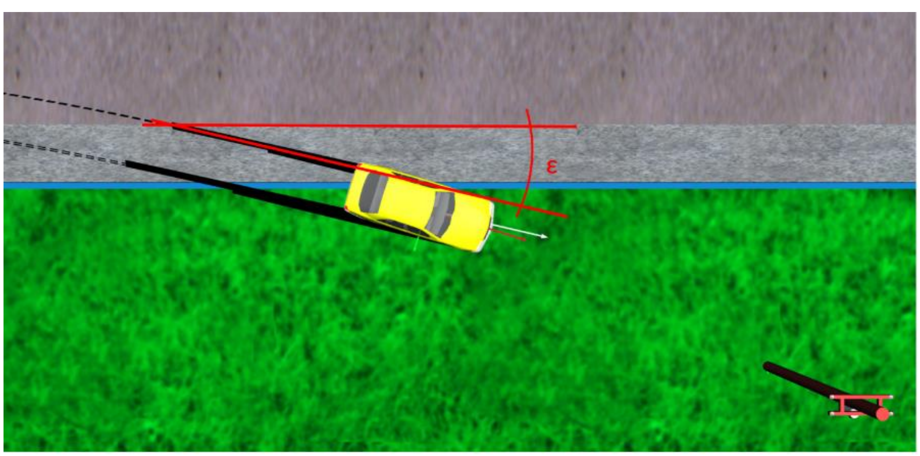


Figure 11. Departure Angle.

Most of the current literature provides varying but consistent ranges for the angle of departure from ROR crashes. For instance, recent studies (Graham et al., 2014; Nash, 2015) reported that the angle of departure ranged from 2.9 to 22 degrees, while Lynam and Kennedy (2005) reported that a majority of the ROR crashes in the United States had an angle of departure varying from 5 to 15 degrees. Another study found an angle of 5 to 25 degrees for the majority of the crashes examined (Doecke et al., 2011), with a maximum of 32 degrees in rare cases. Although results were not stated in terms of angle of departure, a study by Moon and Mihailidis (2013) used the angle of departure that varied from 9 to 19 degrees for computing the clearance zone distance. On average, the angle of departure was reported to be 11 and 12 degrees in a study by Lynam and Kennedy (2005), 14 degrees in two other studies (Doecke & Woolley, 2011; Graham et al., 2014), and 16.9 degrees in a study by Albuquerque and Sicking (2010). The angle is said to vary per ADT, as reported in a study by Graham et al. (2014). According to this study, as the ADT increases from 2,000 to 6,000, the angle of departure increases from 9 to 14 degrees. Contrary to most of the aforementioned studies, one study (Albuquerque et al., 2010) reported a very wide range of angles of departure. That study found that the minimum angle of departure was 0 degrees, while the maximum was 84 degrees. However, the mean (16.9 degrees) and median (15 degrees) for the angle of departure in this study were relatively similar to the rest of the studies cited. Therefore, the research team adopted a range for the angle of departure between 10 and 25 degrees for the purpose of this study.

Equivalent Length of Influence Calculations

Considering the adopted range for angle of departure from 10 degrees up to 25 degrees, the equivalent length of influence is calculated as follows:

$$Equivalent\ length = MaxLength_{object} + \left(Lateral.Ofset_{Object}\right) * [\cot(10^0) - \cot(25^0)]$$

Where cot is the cotangent function of an angle defined in degrees. In the case of pole and lone trees, the maximum length of the object equals the measured diameter or width of the object. For clusters of trees, the maximum length of the object is equal to the maximum length of the cluster of trees plus the average diameter or width. Figure 12 shows a schematic of the definition of equivalent length of influence for a cluster of trees.

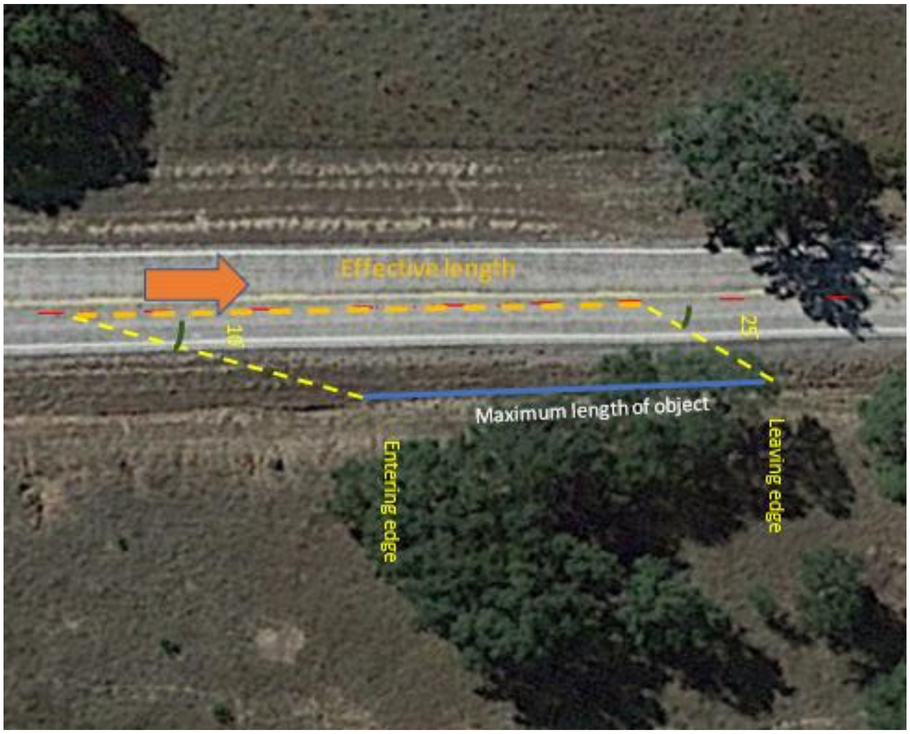


Figure 12. Equivalent Length of Influence Example for a Cluster of Trees.

As can be seen in Figure 12, the equivalent length of influence associated with a roadside object is greater than the maximum length of the object in general. In terms of the kinematics of a departing vehicle, it is expected that speed of departure and side-slope grade should play a role in defining the equivalent length of influence of an object more accurately. However, the definition above was adopted as an acceptable approximation to consider this complex feature of roadway departure crashes.

Horizontal Curve Summary Variables at the Segment Level

Accounting for horizontal curvature of segments is clearly of interest for this research given the increased risk of roadway departure at these locations. From the data collected, the number of curves, radii, and chord lengths of all horizontal curves within a segment were determined. Using curve radii and chord lengths, the research team determined (a) the percentage composition (by length) of curves within a segment; (b) the lengths, radii, and positions of the sharpest and flattest curves in a segment; and (c) the difference in length between the sharpest and flattest curves.

Naturally, the number of curves was determined by counting the curves in each segment. The percentage share of the curves within a segment was computed by summing up the lengths of all curves in a segment and dividing the total by the segment length. Lengths, radii, and positions of the sharpest and flattest curves in a segment were defined as well. The sharpest and flattest curves were determined by using the curve radius for this task. For each segment, the curve with the smallest radius was considered as the sharpest curve. Likewise, the curve with the largest radius was considered as the flattest curve. In addition, the angles and lengths of the identified shortest and longest curves were recorded. The position of the flattest and the sharpest curves were determined with respect to the extremes of the segments. An indicator variable was created to record whether the flattest or the sharpest curves were on the extremes of each segment. The difference in length between the sharpest and flattest curves was also recorded for each segment.

Database Reduction during Data Collection Activities

As anticipated, it was unavoidable to discard some of the sampled segments for various reasons that became apparent during the data collection. Table 9 shows a summary of those issues and the number of segments discarded accordingly. Because of the discarded segments shown in the table, the total number of segments in the final database reduced from 600 to 463 at this stage.

Table 9. Summary of Discarded Segments with Issues.

Issue	Action	Number of Segments
Segment length shorter than 100 ft	Discarded	98
Uneven cross-section in the segment (merging lanes, tapering median width, changing lane widths)	Discarded	23
Multiple lanes in one or both directions	Discarded	16
Total Segments Discarded		137

Final Dataset Overview

The research team performed QC checks upon the completion of data collection, which yielded an initial set of 463 segments. Further revision found additional segments unfit for analysis due to various reasons, including the absence of necessary roadway imagery or segments being too short for analysis. Therefore, the total number of segments further reduced to 422 segments for analysis, which constitutes the final database to support a systemic analysis.

The data collected from aerial-view-based imagery included segment length, lane width, number of curves, guardrail length and distance from paved shoulder, paved shoulder width, number of minor intersections, and number of driveways. Table 10 shows the summary statistics of these features. The average segment length was 0.72 mi, while the minimum and maximum lengths of the segments were 0.01 mi and 6.34 mi, respectively. A total of 321 segments were found to have paved left and right shoulders. The width of paved shoulders varied between 0.14 and 22.66 ft. On average, the paved shoulder width was 5.23 ft in the final database. Guardrails were found for 130 segments. The sum of lengths of guardrails per segment varied between 1.82 ft to 3285.3 ft, with an average of 519.67 ft. Subsets of 276 and 206 segments were found to have driveways and minor intersections, respectively. About 9.16 percent (38 segments) were found to have shoulder rumble strips, while a relatively similar percentage (11.57 percent) of segments were found to have center rumble strips.

Table 10. Summary of Features Collected through Aerial View.

	Number of segments	Minimum	Average	Median	Maximum
Segment length (mi)	422	0.01	0.72	0.29	6.34
Lane width (ft)	422	8.60	11.49	11.53	24.56
Left shoulder width (ft)	321	0.14	5.36	4.33	22.66
Right shoulder width (ft)	321	0.24	5.10	4.32	21.14
Guardrail length (ft)	130	1.82	519.44	370.48	3,285.30
Driveways (n)	276	1.00	5.07	3.00	39.00
Minor intersections (n)	206	1.00	1.72	1.00	7.00

It was found that the speed limit varied between 30 mph and 75 mph, with most of the sections (126) having 55 mph. Curve delineators were observed in 81 segments, while chevrons were found in 20 segments. Summaries can be found in Table 11.

Table 11. Summary of Speed Limit, Chevrons, and Delineators.

Variable	Category	Number of segments	Percent
Speed limit (mph)	30	12	2.93%
	35	8	1.96%
	40	5	1.22%
	45	18	4.40%
	50	11	2.69%
	55	126	30.81%
	60	76	18.58%
	65	18	4.40%
	70	79	19.32%
	75	56	13.69%
Chevrons	0	395	95.18%
	1	4	0.96%
	2	16	3.86%
Delineators	0	334	80.48%
	1	11	2.65%
	2	70	16.87%

Furthermore, the average number of poles, lone trees, and clusters of trees per segment were 11.50, 3.81, and 3.58, respectively. Poles were observed in 217 segments, lone trees in 100 segments, and clusters of trees in 67 segments. Clusters of trees were divided into two types: the first group was clusters that were shorter than 200 ft long, while the second group included clusters that were greater than 200 ft. The longest clusters for each group were identified. The maximum length of clusters within a segment that were shorter than 200 ft varied between 39 ft to 199 ft, while the maximum varied between 231.5 and 2593.1 ft for clusters greater than 200 ft. For clusters shorter than 200 ft, the total number of trees per cluster were counted in case of a segment having three clusters at most, while only trees from three clusters randomly selected were counted and averaged for segments with more than three clusters. It was found that, on average, each cluster had about five trees, with a range from 2 to 12 trees. The effective distance for each cluster, lone tree, and pole were computed consistently with the discussion earlier in this chapter. The results are presented in Table 12.

Table 12. Summary of Poles, Lone Trees, and Clusters of Trees.

Object	Variable	No. of segments	Min.	Avg.	Med.	Max.
Poles	Number of poles	217	1.00	11.50	5.00	108.00
	Average distance to a pole (ft)	217	1.67	33.42	34.24	49.52
	Effective length for poles (ft)	217	6.63	118.66	121.57	175.48
Lone trees	Number of lone trees	100	1.00	3.81	2.00	30.00
	Minimum distance to a tree (ft)	100	6.42	29.46	28.24	49.80
	Average distance to a tree (ft)	100	6.42	33.48	35.46	49.80
	Effective length for trees (ft)	100	24.65	120.13	127.13	177.72
Clusters	Number of clusters	67	1.00	3.58	1.00	20.00
	Average distance to a cluster (ft)	67	9.32	30.76	30.32	49.62
	Average length of a cluster (ft)	67	39.02	290.19	163.50	1,507.20
	Average number of trees per cluster	50	2.00	5.19	5.00	12.00
	Maximum length of a cluster (ft) < 200	50	39.02	116.83	120.17	199.74
	Effective length for clusters (ft) < 200	65	36.73	198.29	205.54	346.5
	Maximum length of a cluster (ft) > 200	32	231.5	722.7	514.3	2,593.1
	Effective length for clusters (ft) > 200	32	455.6	994.6	781.7	2,914.7

Based on the Google Street View images, the research team measured side slopes, guardrail heights, and end terminal widths and heights. The steepest and flattest segments were found to have an average of 41.34 percent and 0.36 percent side slopes, respectively. The guardrail heights from the paved surface averaged about 2.2 ft. Most of the end terminals were found to be rectangular, angled into the ground, or square shaped. See Table 13.

Table 13. Side Slopes and Guardrail Characteristics.

	Number of segments	Minimum	Average	Median	Maximum
Maximum side slope (%)	348	0.36	13.17	10.98	41.34
Guardrail height (ft)	127	0.03	2.23	2.19	3.50
End terminal width (ft)	68	0.73	1.57	1.57	2.80
End terminal height (ft)	69	1.19	1.93	1.96	2.69

A total of 172 segments were found to have horizontal curves (see Table 14). Among 172 segments, 95 segments had one curve per segment, 41 had two curves per segment, and the remaining 36 segments had three or more curves per segment. The number of curves per segment varied from 1 to 12, with an average of 2.17. On average, the curves had a 2,810 ft radius. For segments with multiple curves, the sharpest and the flattest curves were selected and reported per segment. The sequential location of each curve by direction of traffic flow was also determined and recorded in the dataset. Most sharp curves in these segments were defined as either first or second curves in eastbound or southbound directions, respectively. The locations of the sharpest and the flattest curves with respect to the extremes of the segments were also determined. It was found that 75 percent and 75.32 percent of the sharpest and the flattest curves, respectively, were located toward the segment extremes. For the sections with multiple curves, the sharpest curve had a radius of 88 ft. In addition to curve radius, the angular deflection and curve length were provided. The longest curve was 4,732 mi, while the shortest was 31 mi. The angular deflection varied from 0.72 degrees to about 147 degrees, with an average of 36.26 degrees.

Table 14. Horizontal Curve Characteristics.

Feature	No. of segments	Min.	Avg.	Med.	Max.
Number of curves	172	1	2.174	1	12
Percentage of curves (%)	172	2.7	36.3	34.1	88.0
Radius (ft)	172	88	2,810	1,410	104,526
Radius of the sharpest curves	77	90	1,571	1,124	11,313
Radius of the flattest curves	77	402	4,028	2,577	23,601
Angle (degrees)	171	0.72	36.26	30.82	146.95
Angle of the sharpest curve	77	0.72	37.06	30.91	146.95
Angle of the flattest curve	77	1.21	23.79	18.47	90.59
Curve length (ft)	171	31	952	829	4,732
Length of the sharpest curve	77	31	709	641	1,891
Length of the flattest curve	77	114	1,084	805	10,829
Difference in length (flattest & sharpest)	77	4	2,454	2,454	23,071

Because the data were collected from a probability sample, the features just described are representative of the statewide conditions in Texas highways, with metrics of uncertainty available. The research team developed preliminary statewide assessments for select roadside characteristics. This preliminary characterization is intended as a check on the level of precision that can be obtained for statewide estimates using the probability sample at hand.

Regarding the level of exposure to roadside objects, the research team calculated the proportion of roadside that is influenced by roadside objects. This is the proportion of roadside that has potential to result in fixed-object collision if a departing vehicle is at angles ranging from 10 to 25 degrees (per the definition of equivalent length of influence presented earlier in this report).

Expressed as a percent, this proportion ranged from 0 to 100 percent, with a mean of 21.6 percent and standard deviation of 27.56 percent. Although guardrails are roadside fixed objects, they are applied as countermeasures for their ability to significantly reduce the severity of ROR crashes. Therefore, the research team performed the estimation without including guardrails in order to better assess the proportion of roadside at greater risk of fixed-object crashes and with a realistic potential for improvement. Considering the sampling design (region-stratified equal probabilities without replacement) jointly with the data collected for the probability sample, the research team constructed a Horvitz-Thompson (HT) estimator for the statewide proportion of roadside potentially exposed to fixed-object collisions, other than guardrails. Expressed as a percent, this proportion ranged from 0 to 100 percent, with a mean of 13.56 percent and standard deviation of 20.88 percent. The HT estimator for the statewide proportion was determined to be 13.56 percent, with a standard error of 0.54 percent. Therefore, the statewide proportion of roadside with potential to improvement regarding fixed-object collisions (not including guardrails) is estimated to be between 12.51 and 14.61 percent with 95 percent confidence. As mentioned before, this estimate includes poles and trees within 50 ft of the paved shoulder but no guardrails. Considering that there are a total of 54,017 centerline miles of rural two-lane undivided highways in Texas, the proportion above translates to a statewide estimate of between 13,519 and 15,788 roadside miles at two-lane rural undivided highways potentially vulnerable to roadside fixed-object collisions (95 percent confidence interval for the HT estimator from the probability sample).

DATA COLLECTION FOR WORK CODE UPDATES

The data collection activities for Task 5 were intended to develop a cross-reference database to allow the evaluation of the safety effectiveness of identified roadway departure countermeasures. The research team used the following data sources to develop the database:

- 9 years (2010–2018) of Crash Records Information System (CRIS) data.
- 2017 RHiNO.
- HSIP Work Codes Tables from TxDOT SiteManager.

Crash Records Information System

In the state of Texas, law enforcement is responsible for documenting and reporting crash information, while TxDOT is responsible for assembling and maintaining this information in a crash database known as the Crash Records Information System. CRIS contains multiple tables that are linked by a common crash designation identification number. These tables summarize information related to the crash, each vehicle (also referred to as a unit), and each person involved in the crash.

Figure 13 shows an illustration of the three levels described from the CRIS data structure.

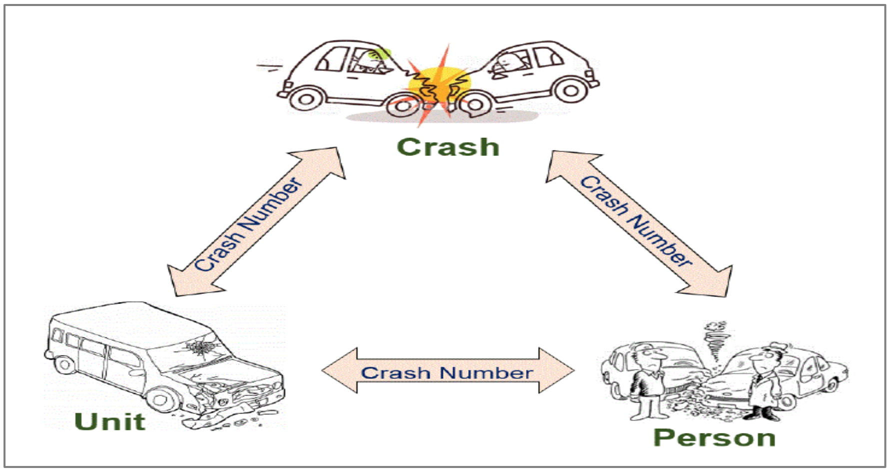


Figure 13. CRIS Data Elements.

Roadway Highway Inventory Network Offload

In addition to the CRIS database, TxDOT also maintains a database that includes a variety of roadway characteristics. This database, known as RHiNO, can be used to supplement information from the crash database.

Figure 14 shows a map of all RHiNO segments available in its 2017 version. Each segment in this figure contains codes that allow uniquely linking each CRIS record to the corresponding segment.

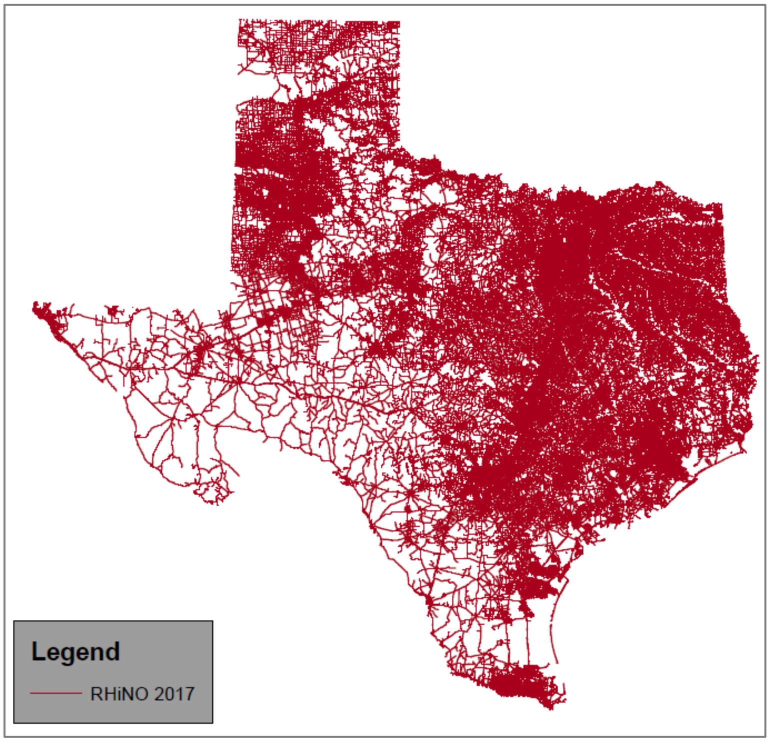


Figure 14. RHiNO Network in 2017.

TxDOT HSIP Work Codes

The research team used roadway codes and associated geographical coordinates to assign crashes to the appropriate roadway segments. The researchers used information and data found in the TxDOT HSIP Work Codes (WCs) Table (Rawson, 2015), which includes 98 WCs that are grouped into five general categories:

- 100 Signing and Signals.
- 200 Roadside Obstacles and Barriers.
- 300 Resurfacing and Roadway Lighting.
- 400 Pavement Markings.
- 500 Roadway Work.

For each WC, the document provides precise description, reduction factor, service life (years), maintenance cost (if available), and preventable crash criteria. The first step of the safety assessment of the work codes is to compile all TxDOT HSIP project data into a master Excel spreadsheet. Table 15 provides a list of potential work codes the research team initially identified for this evaluation.

Table 15. Initial List of Potential Work Codes for Evaluation.

Work Code	Treatment	Potential for Evaluation
204	Flatten Side Slope	High
205	Modernize Bridge Rail and Approach Guardrail	High
206	Improve Guardrail to Design Standards	High
209	Safety Treat Fixed Objects	High
207	Install Protection	Mid
217	Install Impact Attenuation System	Mid
503	Widen Paved Shoulder (to 5 ft or less)	Mid
504	Construct Paved Shoulders (1–4 ft)	Mid
532	Texturize Shoulders (rolled in or milled in)	Mid
533	Texturize Shoulders (Profile Pavement Markers)	Mid
536	Widen Paved Shoulders (to > 5 ft)	Mid
537	Construct Paved Shoulders (5 ft)	Mid
542	Centerline Texturing	Mid
222	Improve Impact Attenuation System	Low
201	Install Median Barrier	Low
222	Improve Impact Attenuation System	Low

After developing the master spreadsheet for data collection, the research team determined the number and percent of missing data in each data attribute for the potential countermeasures listed in Table 15. Missing construction dates in SiteManager and missing before or after crash data were the key reasons found that precluded the research team from including a countermeasure in the master spreadsheet. Table 16 lists the final selected WCs that the research team deemed to have enough data for analysis. The segment length of a road is a crucial component in the development of safety performance functions (SPFs) or CMFs.

The HSM recommends the use of homogeneous segments with respect to AADT, number of lanes, and other associated variables (AASHTO, 2010). Although it is widely accepted that a maximum of 2 mi is appropriate, there is no prescribed minimum segment length for application of the predictive models. However, the literature suggests a segment length longer than 0.10 mi should be appropriate. Miaou and Lum (1993) suggested that short sections less than or equal to 0.05 mi could create bias in the estimation of linear models. Similarly, Ogle et al. (2011) demonstrated that short segment lengths less than 0.10 mi yield uncertain results in crash analysis.

The Texas RHiNO data show a wide variance in segment lengths. To reduce the crash rate bias, the research team decided to conduct a re-segmentation of the roadway data, per the recommendations in the literature listed above. By keeping 0.1 mi as a minimum segment length threshold, the roadways were re-segmented if the RHiNO segment length was found to be over 2 mi, which is another common maximum threshold recommended in the literature. Table 16 shows the summary statistics of the re-segmented segments, as just described. The segments in the RHiNO database exhibit a wide variety of segment lengths. To reduce the potential of estimation bias, the research team decided to conduct re-segmentation of the roadway data, per the recommendations in the literature listed above.

Table 16. Selected Work Codes with Complete Data for Analysis.

Work Code	Description	Original Segments	Total Segment Length (mi)	Re-segmented Segments
206	Improve Guardrail to Design Standards	2	27.3	15
209	Safety Treat Fixed Objects	36	358.9	196
504	Construct Paved Shoulders (1–4 ft)	5	7.1	5
532	Texturize Shoulders (Rolled In or Milled In/Profile Pavement Markers)	7	68.7	37
536	Widen Paved Shoulders to > 5 ft	1	25.8	13
206, 209	Improve Guardrail to Design Standards, Safety Treat Fixed Objects	8	138.3	74
Total		59	625.9	340

By keeping 0.1 mi as a minimum segment length threshold, the roadways were re-segmented if the length of RHiNO segments exceeded 2 mi, per the recommendations above. Table 16 shows the summary statistics of the re-segmented segments as just described.

Data Integration

Data integration work was necessary to merge data obtained from different sources into a single, coherent database for analysis. The flowchart of the data integration steps is shown in Figure 15. The steps of data integration performed by the research team are described next.

- Process 1: Develop a master spreadsheet with construction dates and other relevant information on the WCs from TxDOT SiteManager. Select the final WCs based on the availability of the construction dates and before and after crash data. Perform re-segmentation on the segments that are larger than 2 mi.
- Process 2: Conflate the RHiNO roadway network on the WC segments to population AADT and other geometric information on WC segments.

- Process 3: Use the ArcMap near function to assign crashes to the relevant segments. Assign crashes in the before and after period based on the construction dates.

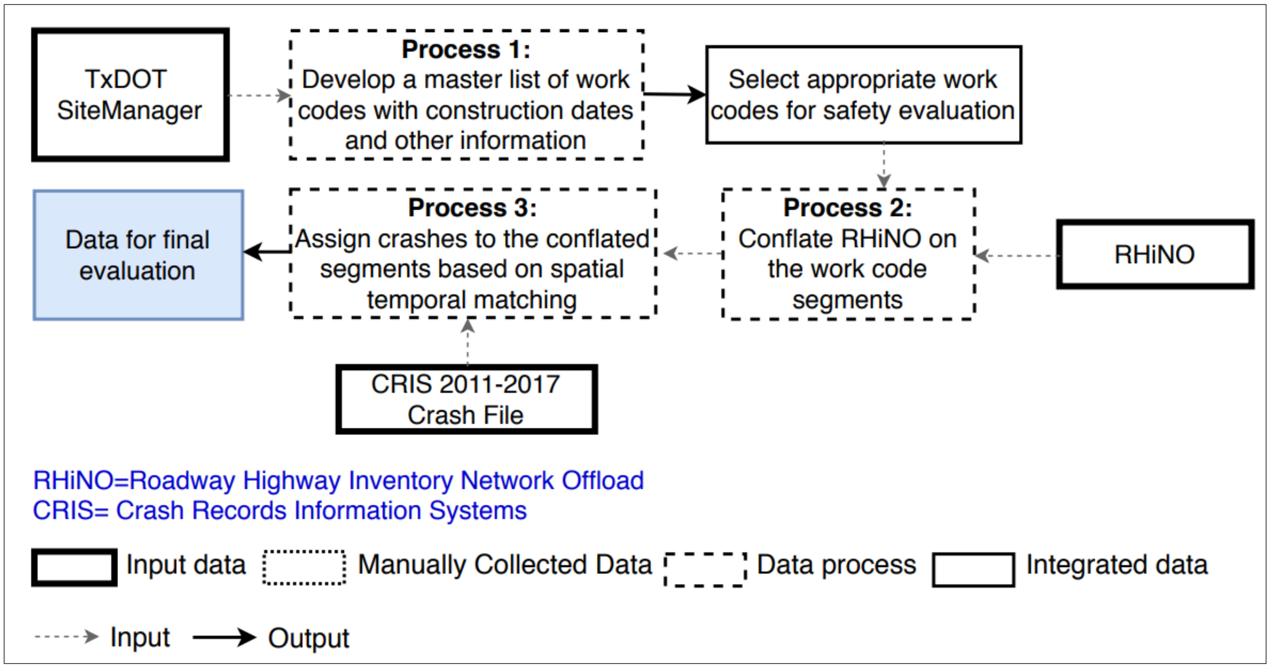


Figure 15. Data Integration Flowchart.

Table 17 provides descriptive statistics of the key variables. The segments with WCs 206 and 532 were found to have higher average AADT values than the segments with different WCs. Another interesting finding was that the average AADTs for the after months tended to be lower than for the before months for all WCs. This trend is counterintuitive.

SUMMARY OF DATA COLLECTION ACTIVITIES

This chapter summarized all activities concerning data collection and preliminary processing for the research project.

To support the systemic analysis, detailed geometric and roadside characteristic data were collected from 463 randomly selected segments representing 328 centerline miles of two-lane undivided highways in Texas.

To support updating roadside countermeasure WCs, the research team identified four WCs with enough data for a thorough analysis (i.e., 8 mi or more of treatment application and sufficient before and after data). Additionally, a big enough sample (195 mi) for the combination of two WCs was identified as well. Because the original segments identified exceeded the recommended threshold of 2 mi for an analysis segment, the research team subdivided each segment to obtain segments at least 0.1 mi long but not to exceed 2.0 mi. This effort resulted in breaking the 78 segments originally identified into 350 new segments for analysis. In total, 921 mi of highway having at least one of the treatments were identified and prepared for analysis.

Table 17. Descriptive Statistics of the Key Variables.

Work Codes	Attribute	Mean	Std. Dev.	Min.	Max.	IQR
206	Segment length (mi)	2.01	0.07	1.84	2.18	0.06
	Lane width (ft)	13.79	2.27	9	16	4
	Months (before)	49.84	3.39	46	56	4
	Months (after)	34.42	15.48	10	52	26
	Avg. AADT (before)	13,199.42	8,471.37	481	24,033	17,820.5
	Avg. AADT (after)	11,749.47	6,832.77	523	20,068	13,905.5
	Total crashes (before)	4.05	2.91	0	9	5
	Total crashes (after)	4.11	4.54	0	21	2.5
209	Segment length (mi)	1.87	0.37	0.1	2.1	0.11
	Lane width (ft)	11.66	1.69	10	19	1
	Months (before)	43.51	16.39	12	59	30
	Months (after)	26.49	18.09	1	59	34
	Avg. AADT (before)	4,955.67	6,839.78	442	50,080	4,420
	Avg. AADT (after)	5,346.77	7,396.58	397	50,840	4,496
	Total crashes (before)	2.85	3.64	0	23	4
	Total crashes (after)	1.69	2.1	0	12	2
504	Segment length (mi)	1.53	0.52	0.72	1.93	0.63
	Lane width (ft)	11	1	10	12	2
	Months (before)	40.4	22.07	12	58	35
	Months (after)	37	20.33	2	51	13
	Avg. AADT (before)	2,557.2	1,559.39	1,542	5,200	1,197
	Avg. AADT (after)	2,822	2,313.56	1,316	6,859	1,133
	Total crashes (before)	4.6	3.85	0	9	6
	Total crashes (after)	8	14.58	0	34	2
532	Segment length (mi)	1.83	0.41	0.46	2.11	0.16
	Lane width (ft)	12.69	1.98	10	17	1
	Months (before)	52.86	3.35	48	59	6
	Months (after)	34.17	12.99	6	53	18
	Avg. AADT (before)	21,873.03	29,944.98	804	126,281	21,844.5
	Avg. AADT (after)	22,745.39	30,649.44	1121	144,036	22,706.25
	Total crashes (before)	8.98	12.92	0	54	9.5
	Total crashes (after)	5.81	7.51	0	35	7
206, 209	Segment length (mi)	1.89	0.4	0.21	2.09	0.09
	Lane width (ft)	11.66	0.85	10	13	0.75
	Months (before)	42.78	17.9	14	59	33
	Months (after)	32.81	16.79	6	55	31
	Avg. AADT (before)	3,067.88	3,277.66	739	19,346	2,200
	Avg. AADT (after)	3,064.31	2,144.75	799	11,642	2,997
	Total crashes (before)	2.41	2.99	0	16	3
	Total crashes (after)	1.86	2.15	0	9	3

CHAPTER 4. SITE PRIORITIZATION USING SYSTEMIC APPROACH

Crashes in rural areas are significantly affected by the random nature of the crash process. This is more prevalent in crash types such as rollovers, guardrail hit crashes, and other fixed-object collisions. Scattered crashes make it much more difficult to efficiently predict or estimate the locations where these crash types will occur. Transportation agencies will continue to experience difficulties when using traditional approaches to implement countermeasures for reducing these rare crash types. Since systemic improvements focus on high-risk roadway features rather than specific locations, it is possible to use the roadway characteristics that are associated with crashes to estimate which locations are most likely to experience the crashes.

The advantages of a systemic approach are noteworthy. A systemic approach needs fewer data once the process is established, and since sites are selected proactively, this approach will help in reducing future crashes. It is important to point out that a systemic approach does not replace the traditional site analysis but instead complements it. While a systemic approach suggests safety treatments based upon roadway system characteristics, the more traditional site analysis suggests safety countermeasures based on operator crash cause and type.

This study proposes a new method based on a systemic approach for reducing the roadway departure crashes and safety-treating guardrails and other fixed objects. The products of this research project will assist TxDOT districts in better prioritizing projects that target roadway departure crashes proactively rather than through a reactive approach, which consequently helps in achieving optimal use of limited resources and maximizing benefits derived from projects implemented as a result.

This chapter has three main sections: (a) a description of the data assembly effort to merge field and crash data; (b) a description of the detailed systemic analysis; and (c) a summary and conclusions for the chapter.

DATA ASSEMBLY

As detailed in Chapter 3, the research team initially identified a total of 600 segments based on a balanced stratified sample across the state of Texas for data collection (the cumulative road length of these segments is 353 mi). After discarding segments due to unavoidable issues related to data collection, the total number of segments in the database reduced from 600 to 420. A detailed description of these segments is presented in Chapter 3.

The selected segments were combined with TxDOT's 2017 RHiNo database to obtain variables such as ADT, truck percentage, shoulder width, lane width, and speed limit. Although the cross-sectional widths and speed limits were collected by the research team from Google Imagery and Street View, respectively, these variables were compared against the values in the RHiNo database. The comparison showed significant differences across two databases, which suggests that it is preferable to collect the data manually to reflect current conditions.

The research team retrieved crash data for the years 2014–2018 from the TxDOT CRIS database. CRIS maintains a statewide automated database for all reported motor vehicle traffic crashes. The data were filtered to include crashes occurring only on main lanes. Only those crashes that were coded as “TxDOT Reportable” were considered. A crash is defined as “TxDOT Reportable” if it occurs on a traffic way and results in an injury or a property damage greater than $1,000.

The crashes are subdivided by the severity of occurrence. The level of injury or property damage due to a crash is referred to as “crash severity.” While a crash may cause a number of injuries of varying severity, the term crash severity refers to the most severe injury caused by a crash. Crash severity is often divided into five categories. The five crash severity levels are:

- K—Fatal injury: an injury that results in death.
- A—Suspected serious injury: any injury, other than a fatal injury, that prevents the injured person from walking, driving, or normally continuing the activities the person was capable of performing before the injury occurred.
- B—Non-incapacitating evident injury: any injury, other than a fatal injury or an incapacitating injury, that is evident to observers at the scene of the crash in which the injury occurred.
- C—Possible injury: any injury reported or claimed that is not a fatal injury, incapacitating injury, or non-incapacitating evident injury and includes claim of injuries not evident.
- O—No injury/property damage only.

SYSTEMIC APPROACH METHOD DEVELOPMENT

This section documents and evaluates the most common characteristics or risk factors for the locations associated with roadway departure crashes. The analysis was carried out on the following crash types: all roadway departure crashes, HO crashes, and fixed-object crashes.

Roadway Departure Crashes

The roadway departure crashes considered in this analysis include ROR and HO collisions. The other opposite directions were found to be negligible and so were not considered. The ROR crashes include guardrail hit crashes, rollovers, and other fixed-object crashes. The research team used the definitions in the Texas SHSP (2017–2022) (TxDOT 2015) for identifying ROR and HO collisions. Table 18 presents the definitions used in the SHSP.

Table 18. Definition of ROR and HO Crashes.

Crash Types and Location	Definition	CRIS Data Codes
Run-Off-Road Crash—All	A single-vehicle crash where the impact of the first harmful event occurred on the shoulder, beyond the shoulder, or in the median of the roadway.	ROAD_RELAT_ID VALUES = 2 – Off Roadway, or 3 – Shoulder, or 4 – Median, and COLLSN_ID = 1 – OMV Vehicle Going Straight, or 2 – OMV Vehicle Turning Right, or 3 – OMV Vehicle Turning Left, or 4 – OMV Vehicle Backing, or 5 – OMV Other
Head-On Crash—All	A crash involving two vehicles going straight that were traveling in opposite directions prior to impact.	COLLSN_ID = 30 – OD Both Going Straight

Table 19 provides the summary statistics of variables that were found to be significant in influencing roadway departure crashes.

Table 19. Summary Statistics of Variables Influencing Roadway Departure Crashes.

Variable	Minimum	Average	Median	Maximum	Total
ADT, vpd	9	1,957	1,080	16,261	—
Segment Length, mi	0.01	0.72	0.29	6.34	302.1
Shoulder Width, ft	0	4.0	2.7	21.9	—
Lane Width, ft	8.6	11.5	11.5	12.0	—
Curve Density, curves/mi	0	1.6	0	43.9	—
Speed Limit, mph	30	60	60	75	—
Roadway Departure Crashes	0	0.8	0	24	355

The following subsections provide more details about the risk factors influencing roadway departure crashes.

Roadway Departure Crash Risk Factors

Different variables were evaluated to identify the risk factors, and the variables presented below were found to be significant in influencing the roadway departure crashes. To identify the risk factors, the research team compared the proportion of crashes for a specific range of a variable with the proportion of existing highway vehicle miles traveled (VMT) (VMT is calculated as a product of segment length and the ADT) within the respective range.

Posted Speed Limit

Figure 16 shows the proportion of roadway departure crashes and the segment VMT as a function of posted speed limit.

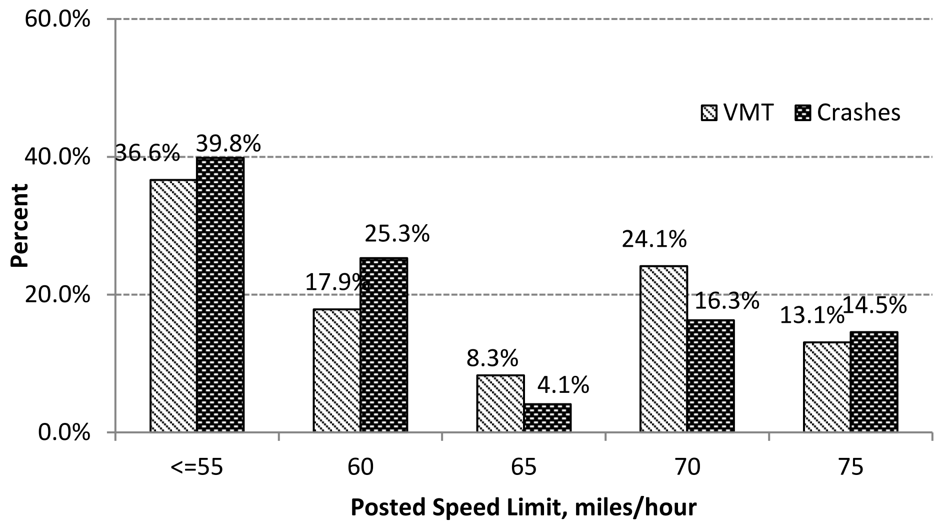


Figure 16. Proportion of Roadway Departure Crashes and VMT as a Function of Speed Limit.

The segments with a posted speed limit of 60 mph or lower and 75 mph have an over-representation of roadway departure crash occurrence.

Curve Density

Figure 17 shows the proportion of roadway departure crashes and the segment VMT as a function of horizontal curve density on the segment. The curve density is calculated by dividing the number of horizontal curves on the segment with its length. As expected, segments with horizontal curves have an over-representation of roadway departure crash occurrence.

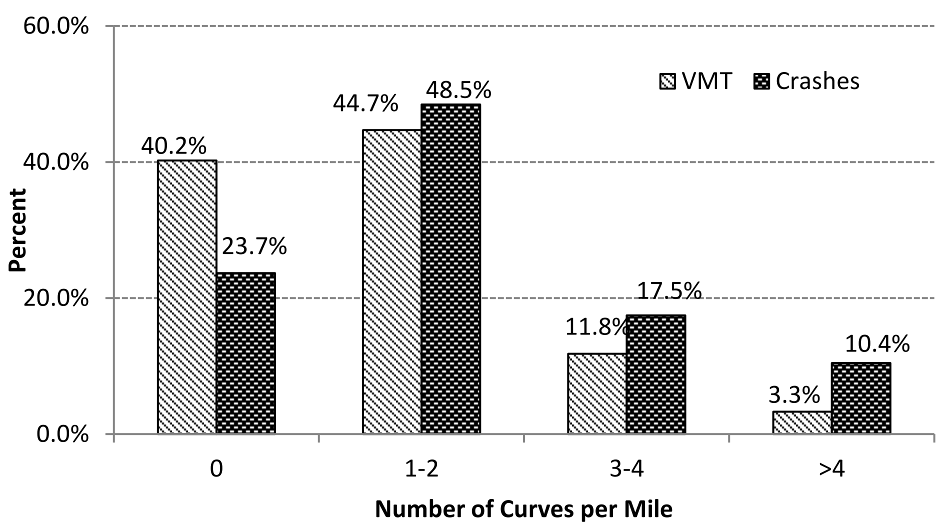


Figure 17. Proportion of Roadway Departure Crashes and VMT as a Function of Curve Density.

Shoulder Width

Figure 18 shows the proportion of roadway departure crashes and the segment VMT as a function of shoulder width. The shoulder width is the average of left and right shoulder widths.

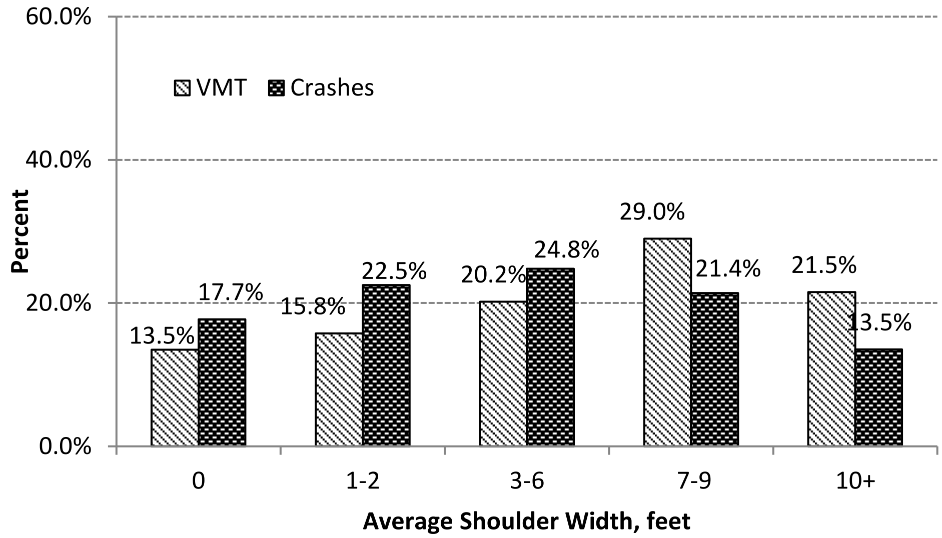


Figure 18. Proportion of Roadway Departure Crashes and VMT as a Function of Shoulder Width.

The segments with a shoulder width less than or equal to 6 ft have an over-representation of roadway departure crash occurrence. This is expected because narrow shoulders provide lesser chance for correction for the vehicle that departed the traveled way.

Lane Width

Figure 19 shows the proportion of roadway departure crashes and the segment VMT as a function of lane width. The lane width is the average width of left and right lane widths. The segments with lane width less than 12 ft have an over-representation of roadway departure crash occurrence. The likelihood of roadway departure crashes increases as the lanes become narrower.

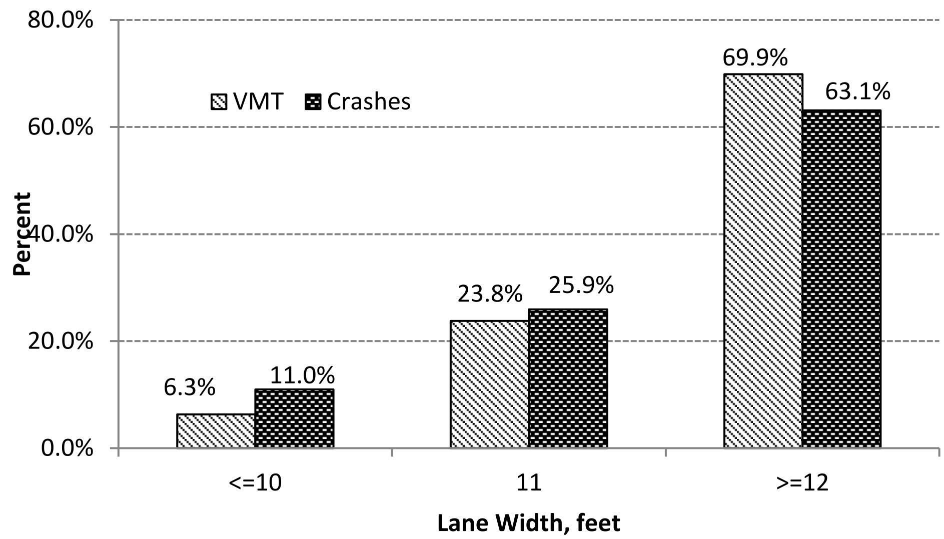


Figure 19. Proportion of Roadway Departure Crashes and VMT as a Function of Lane Width.

Roadway Departure Crash Risk Assessment

In the risk assessment, sites are prioritized using risk factor weights. Risk factor weights are calculated using the total crashes and the crash over- or under-representation of each element. The total risk factor weight is the sum of all risk factor weights of a segment for each element evaluated. Table 2 provides the weights based on the proportion of crash total and crash over-representation when compared to the roadway VMT (Walden et al., 2015).

Based on the weights provided in Table 2, the total weight for a particular risk factor can be calculated using the following equation.

$$W_t = 10 + CT + CO - CU \tag{2}$$

Where,
W_t = total weight;
CT = weight based on crash total;
CO = weight based on crash over-representation; and
CU = weight based on crash under-representation.

Table 20 summarizes the results of risk factor prioritization related to roadway departure crashes on two-lane rural highways. Once the total risk factor weights are evaluated, it is important to understand how the risk factor weights correlate to the crash occurrence. A crash rate was calculated for each segment using the following formula, which considers the years of data, annual traffic, and segment length.

$$Crash\ rate = \frac{Number\ of\ crashes \times 1{,}000{,}000}{Number\ of\ years \times Length \times 365 \times ADT} \tag{3}$$

Table 20. Roadway Departure Crash Risk Factor Prioritization Results.

Risk Factor		Weight (points)
Speed (mph)	<=55	16
	60	19
	65	6
	70	4
	75	12
Curve Density (curves per mile)	0	2
	1–2	17
	3–4	16
	>4	18
Shoulder Width* (ft)	0	15
	1–2	18
	3–6	16
	7–9	5
	>=10	3
Lane Width* (ft)	<=10	15
	11	14
	>=12	10

* Lane and shoulder widths need to be rounded if necessary (e.g., 9.4 ft should be rounded to 9 ft, and 10.5 ft to 11 ft).

The crash rate is compared against the risk factor weight points using the scatterplots and is shown in Figure 20. A linear trend line is fitted and shows that the crash rate increases with the increase in risk factor weights.

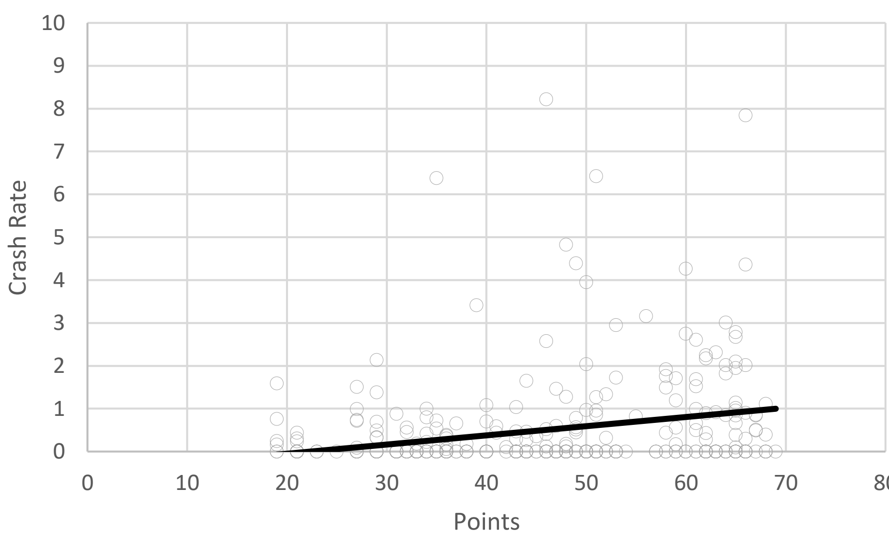


Figure 20. Roadway Departure Crash Rate versus Risk Factor Points.

The next step is to know when a segment can be identified as high risk for roadway departure crashes. Table 21 shows the total weights for different percentiles. For example, a segment with 66 weight points is considered to be in the top 5 percentile high-risk segments in Texas for roadway departure crashes.

Table 21. Roadway Departure Crash Risk Weights Based on Percentiles.

Percentile	Weight (points)
95%	66
85%	63
75%	58
50%	47
25%	35

Guardrail Hit Crashes

The guardrail hit crashes include those that were coded in CRIS as single-vehicle ROR and hit either guardrail, bridge rail, or concrete traffic barrier. Segments with no guardrails were excluded from this analysis. The number of segments reduced from 420 to 129. Table 22 provides the summary statistics of variables found to be significant in influencing the guardrail hit crashes.

Table 22. Summary Statistics of Variables Influencing Guardrail Hit Crashes.

Variable	Min.	Avg.	Med.	Max.	Tot.
ADT, vpd	30	2,597	1,794	11,953	—
Segment Length, miles	0.01	0.74	0.35	5.57	95.71
Speed Limit, mph	30	62	60	75	—
Guardrail Offset, ft	0	0.9	0	17.4	—
Guardrail Hit Crashes	0	0.2	0	4	30

Guardrail Crash Risk Factors

This section documents and evaluates the most common characteristics or risk factors for the locations associated with guardrail hit crashes. Different variables were evaluated to identify the risk factors, and the variables presented below were found to be significant in influencing the guardrail hit crashes.

Posted Speed Limit

Figure 21 shows the proportion of guardrail hit crashes and the segment VMT as a function of posted speed limit. The segments with a posted speed limit of 70 mph or higher and 60 mph have an over-representation of guardrail hit crash occurrence.

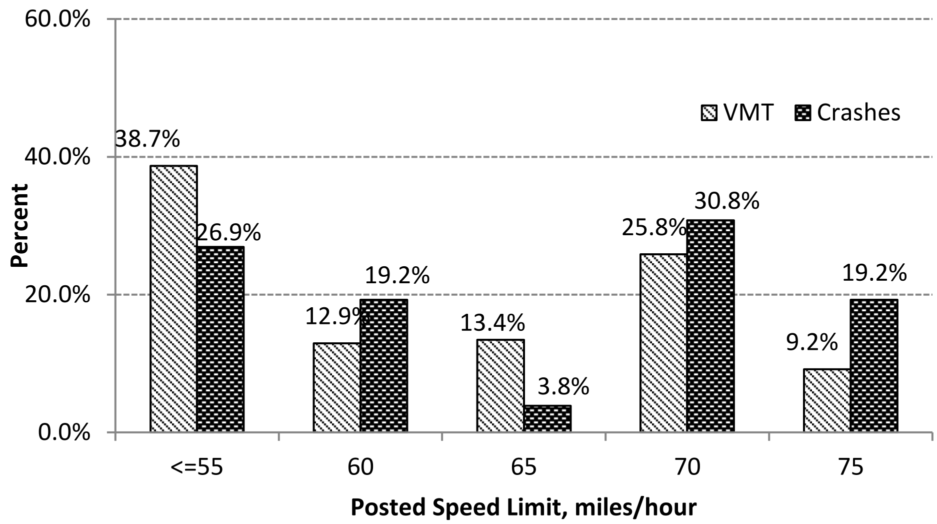


Figure 21. Proportion of Guardrail Hit Crashes and VMT as a Function of Speed Limit.

Guardrail Offset

Figure 22 shows the proportion of guardrail hit crashes and the segment VMT as a function of guardrail offset from the paved surface. The offset is measured from the edge of the paved surface and does not include the paved shoulders. The segments that have guardrails on the edge of the paved surface have an over-representation of guardrail hit crashes when compared to segments that have guardrails more than 1 ft from the paved surface.

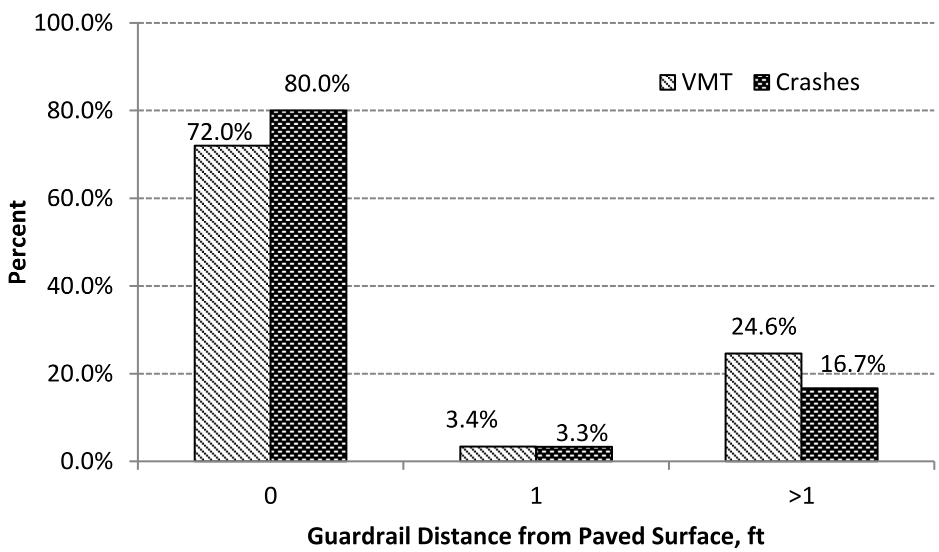


Figure 22. Proportion of Guardrail Hit Crashes and VMT as a Function of Offset.

Table 23 summarizes the results of risk factor prioritization related to guardrail crashes on two-lane rural highways.

Table 23. Guardrail Hit Crash Risk Factor Prioritization Results.

Risk Factor		Weight (points)
Speed (mph)	<= 55	2
	60	17
	65	1
	70	17
	75	21
Guardrail Offset (ft)	0	26
	0.1–1	12
	>1	4

The guardrail hit crash rate is compared against the risk factor weight points using the scatterplots and is shown in Figure 23. A linear trend is fitted and shows that the crash rate increases with the increase in risk factor weights.

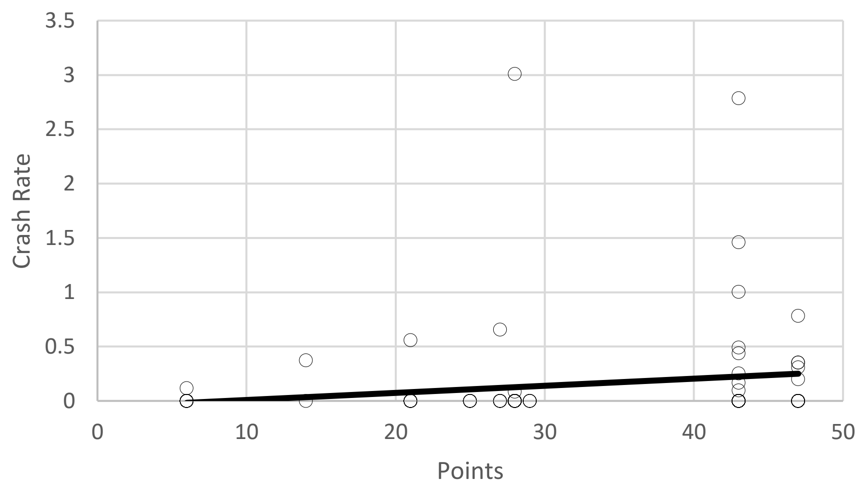


Figure 23. Guardrail Hit Crash Rate versus Risk Factor Points.

The next step is to know when a segment can be identified as high risk for guardrail hit crashes. Table 24 shows the total weights for different percentiles.

Table 24. Guardrail Hit Crash Risk Weights Based on Percentiles.

Percentile	Weight (points)
85%	47
75%	43
50%	28
25%	27

Fixed-Object Crashes

The fixed-object crashes include those that were coded in CRIS as single-vehicle ROR and hit roadside fixed objects other than guardrail, bridge rail, or concrete traffic barrier. Rollover crashes were excluded from the analysis. Segments that have guardrails throughout were not included in this analysis. The number of segments reduced from 420 to 227. Table 25 summarizes the fixed-object crashes by severity on the selected segments.

Table 25. Summary Statistics of Variables Influencing Fixed-Object Crashes.

Variable	Min.	Avg.	Med.	Max.	Tot.
ADT, vpd	9	1,910	996	9,651	Not applicable
Segment Length, mi	0.01	1.00	0.50	6.34	227.64
Shoulder Width, ft	0	3.4	1.9	21.9	Not applicable
Lane Width, ft	8.7	11.4	11.5	12.0	Not applicable
Curve Density, curves/mi	0	1.7	0.4	43.9	Not applicable
Speed Limit, mph	30	58	55	75	Not applicable
Clear Zone Width, ft	1.7	33.2	34.1	49.1	Not applicable
Fixed-Object Crashes	0	0.7	0	21	165

Fixed-Object Crash Risk Factors

This section documents and evaluates the most common characteristics or risk factors for the locations associated with fixed-object crashes. Different variables were evaluated to identify the risk factors, and the variables presented below were found to be significant in influencing the fixed-object crashes.

Posted Speed Limit

Figure 24 shows the proportion of fixed-object crashes and the segment VMT as a function of posted speed limit. Similar to roadway departure crashes, the segments with a posted speed limit of 60 mph or lower and 75 mph have an over-representation of fixed-object crash occurrence.

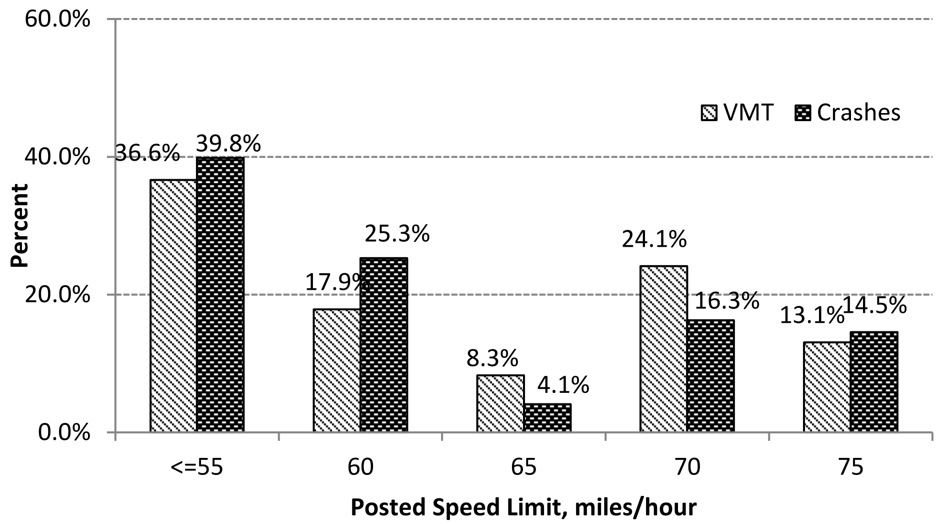


Figure 24. Proportion of Fixed-Object Crashes and VMT as a Function of Speed Limit.

Curve Density

Figure 25 shows the proportion of fixed-object crashes and the segment VMT as a function of horizontal curve density on the segment. Segments with one or more horizontal curves have an over-representation of fixed-object crash occurrence.

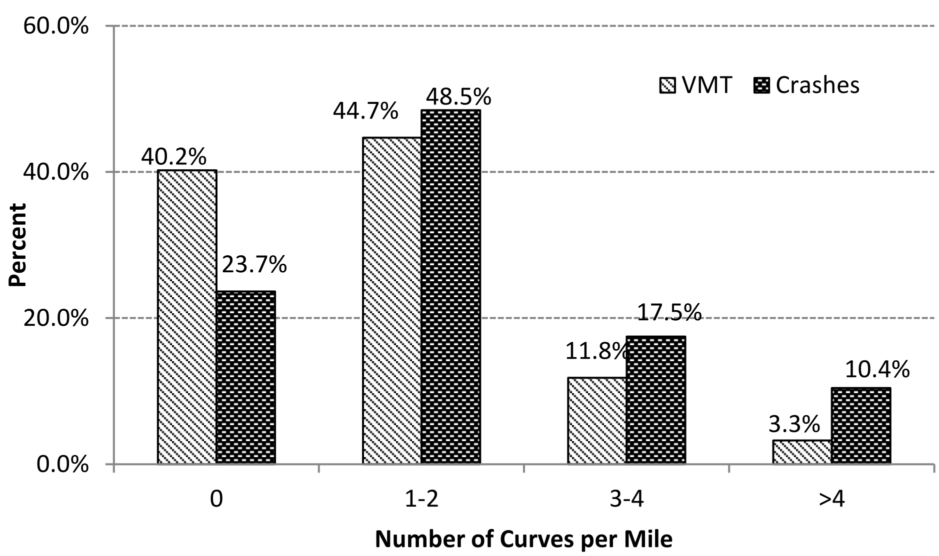


Figure 25. Proportion of Fixed-Object Crashes and VMT as a Function of Curve Density.

Shoulder Width

Figure 26 shows the proportion of fixed-object crashes and the segment VMT as a function of shoulder width. The segments with a shoulder width less than or equal to 6 ft have an over-representation of fixed-object crash occurrence. This is expected because narrow shoulders provide lesser chance for correction for the vehicle that departed the traveled way, which consequently may collide with a roadside fixed object.

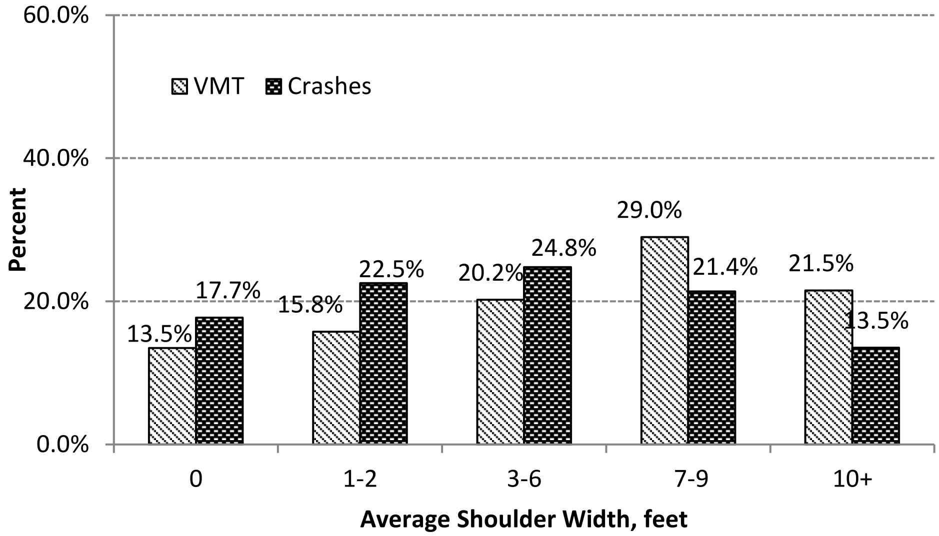


Figure 26. Proportion of Fixed-Object Crashes and VMT as a Function of Shoulder Width.

Lane Width

Figure 27 shows the proportion of fixed-object crashes and the segment VMT as a function of lane width. The segments with lane width less than 12 ft have an over-representation of fixed-object crash occurrence.

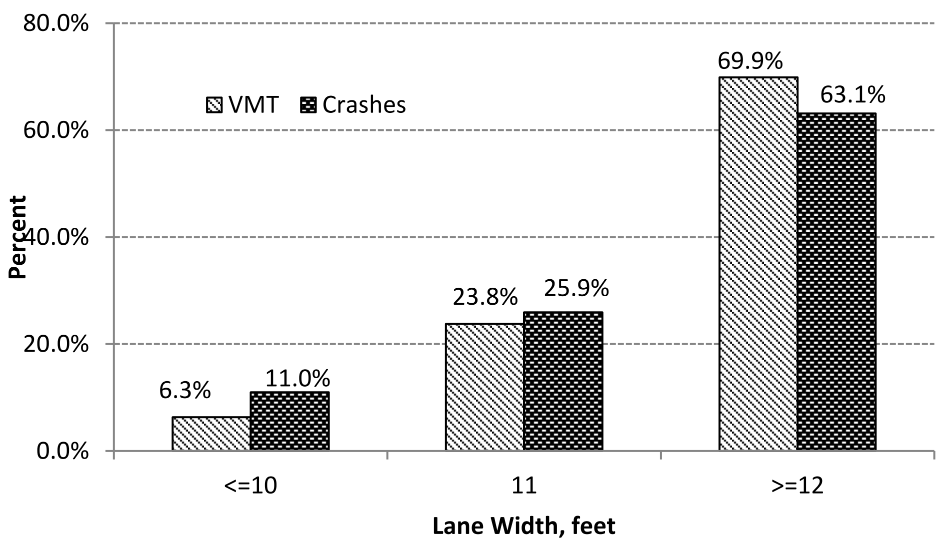


Figure 27. Proportion of Fixed-Object Crashes and VMT as a Function of Lane Width.

Clear Zone Width

Figure 28 shows the proportion of fixed-object crashes and the segment VMT as a function of clear zone width. The clear zone width is measured from the edge of the traveled way to the nearest continuous line of vertical objects (i.e., tree line, fence line, or utility poles) that are roughly parallel to the road centerline. The ROR crash frequency and severity will be reduced by increasing the lateral offset to vertical obstructions along the roadside. It is often not possible to relocate or remove objects from the clear zone. In such cases, objects are protected by barrier or made to operate in a break-away manner. As shown in Figure 28, the segments with clear zone width less than 30 ft have an over-representation of fixed-object crash occurrence.

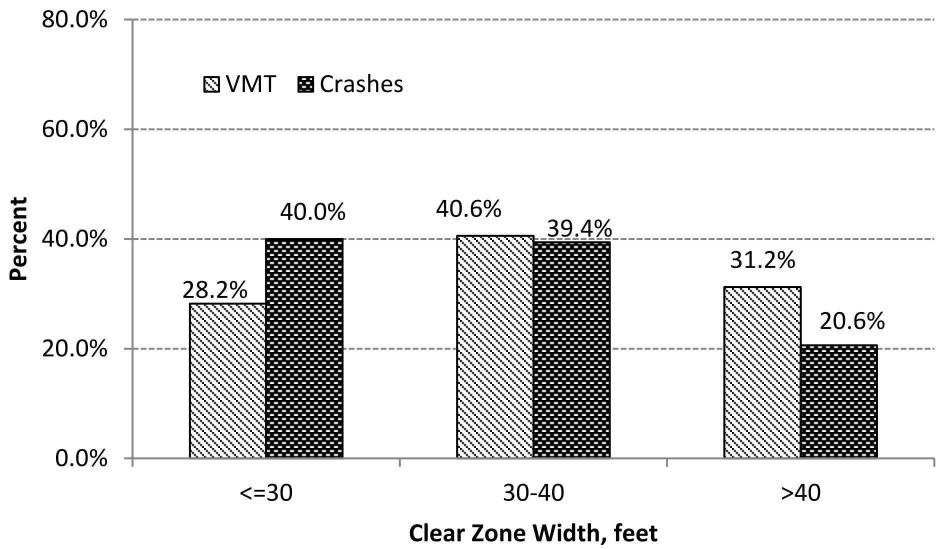


Figure 28. Proportion of Fixed-Object Crashes and VMT as a Function of Clear Zone Width.

The research team acknowledges that TxDOT may have difficulties in obtaining the clear zone width. In such cases, the approximate widths provided in Table 26 may be used. These widths were obtained based on the right-of-way (ROW) width values provided for each segment in the RHiNo database.

Table 26. Clear Zone Widths as a Function of ROW Widths.

ROW, ft	Clear Zone Width, ft
<=70	20
70–80	30
>80	40

Fixed-Object Risk Assessment

Table 27 summarizes the results of risk factor prioritization related to fixed-object crashes on two-lane rural highways.

Table 27. Fixed-Object Crash Risk Factor Prioritization Results.

Risk Factor		Weight (points)
Speed (mph)	<=55	22
	60	18
	65	6
	70	1
	75	10
Curve Density (curves per mile)	0	2
	1–2	21
	3–4	15
	>4	18
Shoulder Width* (ft)	0	14
	1–2	21
	3–6	20
	7–9	1
	>=10	1
Lane Width* (ft)	<=10	12
	11	14
	>=12	11
Clear Zone Distance (ft)	<=30	24
	30–40	12
	>40	2

* Lane and shoulder widths need to be rounded if necessary (e.g., 9.4 ft should be rounded to 9 ft, and 10.5 ft to 11 ft).

The fixed-object crash rate is compared against the risk factor weight points using the scatterplots and is shown in Figure 29. A linear trend is fitted and shows that the crash rate increases with the increase in risk factor weights.

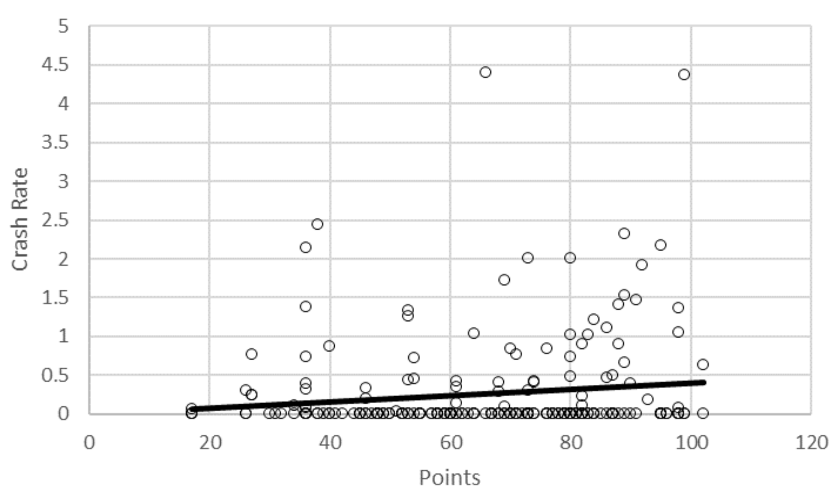


Figure 29. Fixed-Object Crash Rate versus Risk Factor Points.

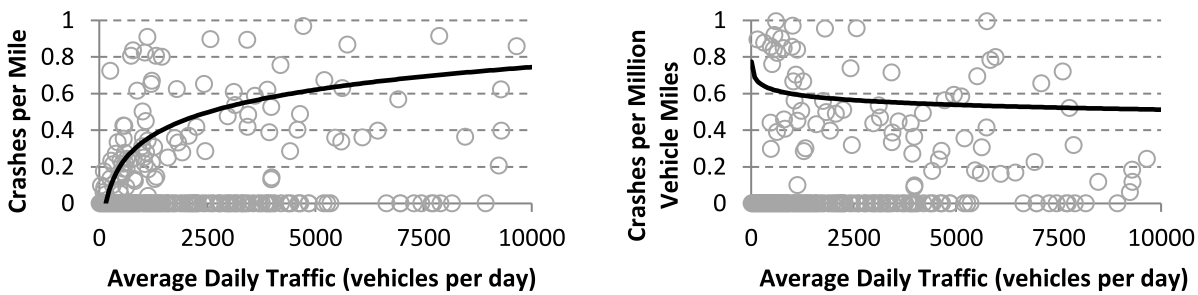
The next step is to know when a segment can be identified as high risk for fixed-object crashes. Table 28 shows the total weights for different percentiles.

Table 28. Fixed-Object Crash Risk Weights Based on Percentiles.

Percentile	Weight (points)
95%	98
85%	87
75%	82
50%	70
25%	52

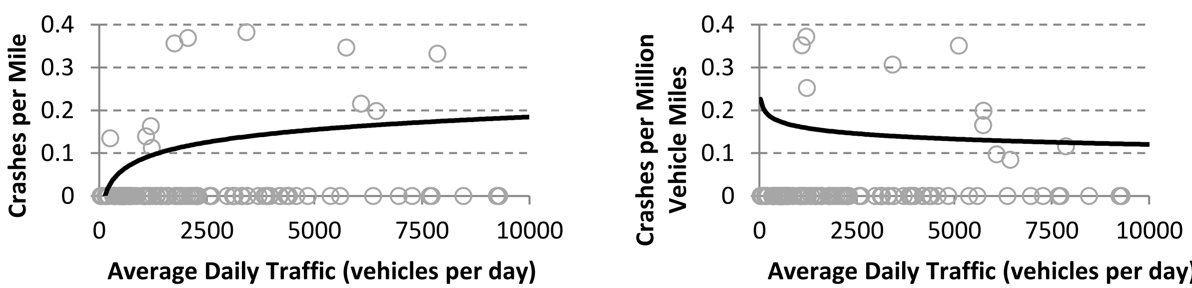
AVERAGE DAILY TRAFFIC CONSIDERATIONS

Figure 30, Figure 31, and Figure 32 depict the relationship of ADT with roadway departure crashes, guardrail hit crashes, and fixed-object crashes, respectively. Figure 30a, Figure 31a, and Figure 32a show the relationship between crashes per mile and ADT, whereas Figure 30b, Figure 31b, and Figure 32b show the relationship between crash rate and ADT. Scatterplots were used for each segment, and a logarithmic trend line is fitted to show the relationships. These figures show that as ADT increases, the number of crashes increases. However, the crash risk per vehicle decreases as the ADT increases.



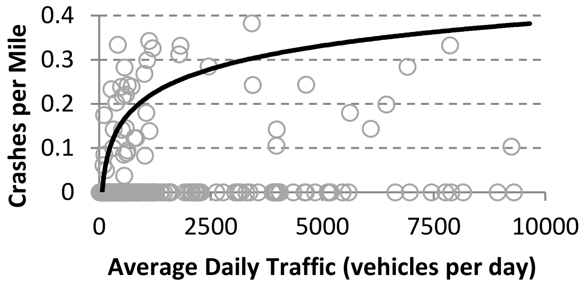
a) Crashes per mile vs ADT　　b) Crash Rate vs ADT

Figure 30. Relationship of Roadway Departure Crashes with ADT.

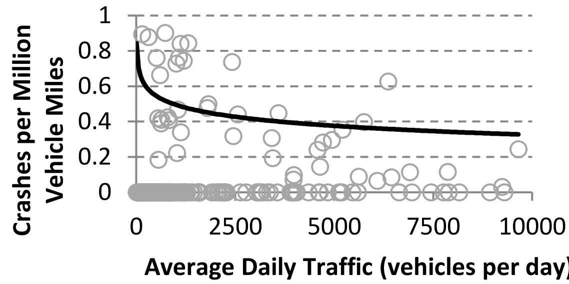


a) Crashes per mile vs ADT　　b) Crash Rate vs ADT

Figure 31. Relationship of Guardrail Hit Crashes with ADT.



a) Crashes per mile vs ADT



b) Crash Rate vs ADT

Figure 32. Relationship of Fixed-Object Crashes with ADT.

It should be noted that the ADT is not considered as a risk factor in the systemic analyses. There are two primary reasons for not considering the ADT. As shown in Table 2, crash totals were used in the risk factor weights. Since crash occurrence is positively correlated to the traffic flow, ADT is indirectly captured when the crash total is considered. Second, if the ADT is used as the risk factor and the proportion of crashes is compared to the proportion of mileage (i.e., comparing the crashes per mile) for each ADT group, then the site selection is inadvertently biased toward the high-volume roadways. This is mainly because crashes are a function of traffic volume, and seeing more crashes at high-volume sites is generally expected. Alternatively, if the proportion of crashes is compared to the proportion of VMT (i.e., comparing the crash rate) for each ADT group, then the site selection is inadvertently biased toward the low-volume roadways. Since reducing the crash frequency and the crash risk is equally important, it is recommended to separate the segments into different volume groups and prioritize the sites in each group for treatment.

SUMMARY OF WORK TO DEVELOP SYSTEMIC APPROACH METHODS

This study developed a method based on a systemic approach for reducing roadway departure crashes and safety-treating guardrails and other fixed objects. In order to perform a systemic analysis of roadway departure crashes in Texas, the research team searched key variables that influence roadway departure crashes. Based on a balanced stratified sample across the state of Texas, the research team initially identified a total of 600 segments for data collection (the cumulative road length of these segments is 353 mi). After discarding a few segments due to unavoidable issues related to data collection, the total number of segments in the database reduced from 600 to 420. The data were collected from TxDOT databases, and an extensive manual data collection was performed as well.

Roadway departures account for the majority of fatal crashes in rural areas. Recognizing this issue, many states, including Texas, identified roadway departure crashes as one of the emphasis areas of their state safety strategic plans. In support of the efforts in Texas to curb roadway departure crashes, this research developed a systemic approach method for prioritizing sites more at risk of roadway departure, guardrail, and fixed-object crashes. Sensible risk factors and prioritization methods were developed and outlined for each crash type. Further details, conclusions, and recommendations from this work are provided in Chapter 6.

CHAPTER 5. SAFETY EVALUATION OF COUNTERMEASURES

This chapter documents the efforts toward updating a subset of work codes that target roadway departure crashes through a safety evaluation of past HSIP projects in Texas. This chapter has three sections: (a) the methodology applied; (b) analysis and results; and (c) conclusions and recommendations.

METHODOLOGY

This section documents the approach used in this study to quantify the effectiveness of RwD treatments (i.e., WCs). Specifically, the next subsection briefly describes the EB method. Later in this section, an additional subsection discusses the SPFs used in implementing the EB approach.

Empirical Bayes Method

A before-after study with the EB approach has been recognized as a robust method for developing CMFs. The EB method is able to account for the regression-to-the-mean bias, account for other changes over time not due to the treatment, and reduce the level of uncertainty in the estimates of safety (Montella 2009).

In most traffic safety before-after studies, the CMF is estimated by comparing the number of expected crashes that would have occurred in the after period had the treatment not been implemented with the number of reported crashes in the after period with the treatment. The EB method uses the safety performance of sites similar to treated sites for estimating the expected number of crashes had the treatment not been implemented. Particularly, it combines the observed number of crashes at a site with the estimated number of crashes of similar sites. The estimation is usually obtained from the use of SPFs.

Many studies have discussed the steps of the EB study and applied the method in estimating the safety effectiveness of treatments (Hauer, 1997; Gross et al., 2010; Lord & Geedipally, 2014). This project mainly followed the procedure in a recent study conducted by Wu et al. (2018), but necessary changes have been made to accommodate the objectives of this project. The steps are described below.

Step 1: Estimate the Expected Number of Crashes in the Before Period

Using an SPF, the predicted number of crashes for a site can be estimated using Equation 4.

$$E\left[\hat{k}_i\right] = t \times \mathrm{C} \times f(ADT, length, other\ roadway\ factors) \tag{4}$$

Where,
$E[\hat{k}_i]$ = predicted number of target crashes (e.g., KAB RwD) for site *i*;
t = duration of the study period (in number of months or years in this study);
C = calibration factor; and
$f(ADT, length, other\ roadway\ factors)$ = safety performance function for a set of site characteristics, such as volume, segment length (mi), and roadway factors.

In this study, the SPFs were developed specifically for roadway departure crashes based on similar roadways as treated sites in Texas (as will be discussed in the next section). Thus, the calibration factor is 1.0.

The EB method estimates the predicted number of crashes before implementation of the countermeasure at each treatment site and the variance of $E[\hat{k}_i|K_i]$. The estimate $E[\hat{k}_i|K_i]$ is calculated by combining the predicted crashes $(E[k]_i)$ with the observed count of crashes (K_i) in the before period, and is given as follows:

$$E[\hat{k}_i|K_i] = \hat{w}_i \cdot E[\hat{k}_i] + (1 - \hat{w}_i) \cdot K_i \tag{5}$$

Where,
$E[\hat{k}_i|K_i]$ = EB estimate of the expected number of crashes for site *i*;
$\hat{w}_i$ = weight factor; and
K_i = observed number of crashes.

The variance of the estimate is given as:

$$Var\left[E[\hat{k}_i|K_i]\right] = (1 - \hat{w}_i) \cdot E\left[\hat{k}_i\middle|K_i\right] \tag{6}$$

Details of SPFs and weight factors are documented in the next section.

Step 2: Calculate the Ratio of the After-Period Crash Estimate to the Before-Period Estimate

With the SPFs used in Step 1, estimate the predicted number of crashes $(E[z_i])$ in the after period at each treatment site. The ratio of the after-period crash estimate to the before-period estimate (P_i) is calculated as:

$$P_i = \frac{E[\hat{Z}_\iota]}{E[\widehat{k_\iota}]} \tag{7}$$

Step 3: Obtain the Estimated Crashes ($\hat{\pi}_i$) and the Estimated Variance

Calculate the estimated crashes during the after period that would have occurred without implementing the countermeasure (a work code treatment in this case).

The estimated number of crashes ($\hat{\pi}_i$) is given by:

$$\hat{\pi}_i = P_i \times E[\hat{k}_i|K_i] \tag{8}$$

The estimated variance of $\hat{\pi}_i$ is given by:

$$Var[\hat{\pi}_i] = P_i \times (1 - \hat{w}_i) \times E\left[\hat{k}_i\middle|K_i\right] \tag{9}$$

Step 4: Compute the Sum of the Estimated and Observed Crashes at All Sites in the Treatment Group

The number of after-period crashes for a group of sites had the treatment not been implemented at the treated sites is given as:

$$\hat{\pi} = \sum_{i=1}^{J} \hat{\pi}_i \tag{10}$$

where J represents the total number of sites in the treatment group, and $\hat{\pi}$ is the estimated after-period crashes at all treated sites had there been no treatment, as described above.

Step 5: Compute the Sum of the Actual Crashes at All Treated Sites

For a treated site, crashes in the after period are influenced by the implementation of the treatment. The safety effectiveness of a treatment is assessed by comparing the actual crashes with the treatment to the estimated crashes without the treatment. The actual number of after-period crashes for a group of treated sites is given as:

$$\hat{\lambda} = \sum_{i=1}^{J} L_i \tag{11}$$

where L_i is the crash frequency during the after period at site i. The estimate of $\hat{\lambda}$ is equal to the sum of the observed number of crashes at all treated sites during the after study period.

Step 6: Estimate $Var[\hat{\lambda}]$ and $Var[\hat{\pi}]$

Based on the assumption of the Poisson distribution, the estimate of variance of $\hat{\lambda}$ is assumed to be equal to L. The estimate of variance of $\hat{\pi}$ can be calculated as follows:

$$Var[\hat{\lambda}_i] = L_i \tag{12}$$

$$Var[\hat{\lambda}] = \sum_{i=1}^{J} Var[\hat{\lambda}_i] \tag{13}$$

$$Var[\hat{\pi}_i] = P_i \cdot (1 - \hat{w}_i) \cdot E[\hat{k}_i | K_i] = (1 - \hat{w}_i) \cdot \hat{\pi}_i \tag{14}$$

$$Var[\hat{\pi}] = \sum_{i=1}^{j} Var[\hat{\pi}_i] \tag{15}$$

Step 7: Compute the Safety Effectiveness of the Treatment

The CMF is estimated as the ratio of what safety was with the treatment to what it would have been without the treatment.

$$\widehat{CMF} = \frac{\frac{\hat{\lambda}}{\hat{\pi}}}{1 + \frac{Var[\hat{\pi}]}{\hat{\pi}^2}} \tag{16}$$

The percent change (known as crash reduction factor, or CRF) in the number of target crashes due to the treatment is calculated by 100(1-$\widehat{CMF}$) percent. If $\widehat{CMF}$ is less than 1, then the treatment has a positive safety effect. The estimated variance and standard error of the estimated safety effectiveness are given by:

$$Var(\widehat{CMF}) = \widehat{CMF}^2 \times \frac{\left(\frac{1}{\hat{\lambda}}+\frac{Var[\hat{\pi}]}{\hat{\pi}^2}\right)}{\left(1+\frac{Var[\hat{\pi}]}{\hat{\pi}^2}\right)} \tag{17}$$

$$s.e.(\widehat{CMF}) = \sqrt{Var(\widehat{CMF})} \tag{18}$$

The approximate 95 percent confidence interval for CMF is given by adding and subtracting $1.96 \times s.e.(\widehat{CMF})$ from $\widehat{CMF}$. If the confidence interval contains the value 1, then no significant effect has been observed.

For the detailed derivation of Equations 5 through 18, please refer to Wu et al. (2018).

Safety Performance Functions and Crash Modification Factors

It is important to note that the EB estimate is essentially a combination of two sources: the prediction from a set of SPFs and CMFs, and the observed number of crashes. The roadway departure safety implementation plan for Texas, supported by FHWA, developed several RwD SPFs for rural roadways (FHWA, 2020). This study used the SPFs developed in the FHWA report for the following facility types:

- Rural two-lane.
- Rural four-lane undivided.
- Rural four-lane divided.

The SPFs from that effort used in this research project are shown in Table 29 through Table 31.

Table 29. SPFs for Rural Two-Lane Roadways without Two-Way Left-Turn Lane.

Source	Range	RwD SPF	Inverse Dispersion Parameter
KABCO Crashes			
Texas	0.1 < Segment Length ≤ 2.0 ADT < 1,000	$RwD\ KABCO\ Crashes = 0.002851 \times ADT^{0.94} \times Segment\ Length^{0.72} \times e^{(-0.03 \times \text{Paved } Width)}$	0.977
	0.1 ≤ Segment Length < 0.8 ADT ≥ 1,000	$RwD\ KABCO\ Crashes = 0.0442 \times ADT^{0.53} \times Segment\ Length^{0.75} \times e^{(-0.03 \times \text{Paved } Width)}$	0.883
	0.8 ≤ Segment Length ≤ 2.0 ADT ≥ 1,000	$RwD\ KABCO\ Crashes = 0.0334 \times ADT^{0.60} \times Segment\ Length^{0.76} \times e^{(-0.03 \times \text{Paved } Width)}$	1.705
KAB Crashes			
Texas	0.1 < Segment Length < 0.66	$RwD\ KAB\ Crashes = 0.0091 \times ADT^{0.59} \times Segment\ Length^{0.76} \times e^{(-0.03 \times \text{Paved } Width)}$	0.392
	0.66 ≤ Segment Length ≤ 2.0 ADT < 610	$RwD\ KAB\ Crashes = 0.00096 \times ADT^{0.88} \times Segment\ Length^{0.63} \times e^{(-0.01 \times \text{Paved } Width)}$	0.404
	0.66 ≤ Segment Length ≤ 2.0 ADT ≥ 610	$RwD\ KAB\ Crashes = 0.0155 \times ADT^{0.55} \times Segment\ Length^{0.77} \times e^{(-0.04 \times \text{Paved } Width)}$	1.109

Table 30. SPFs for Rural Four-Lane Undivided Roadways.

Source	Range	RwD SPF	Inverse Dispersion Parameter
KABCO Crashes			
Texas	0.1 < Segment Length ≤ 0.82	$RwD\ KABCO\ Crashes = 0.0367 \times ADT^{0.51} \times Segment\ Length^{0.79} \times e^{(-0.01 \times \text{Paved } Width)}$	1.201
	0.82 < Segment Length ≤ 2.0	$RwD\ KABCO\ Crashes = 0.1511 \times ADT^{0.40} \times Segment\ Length^{0.42} \times e^{(-0.01 \times \text{Paved } Width)}$	1.923
KAB Crashes			
Texas	0.1< Segment Length ≤ 2.0	$RwD\ KAB\ Crashes = 0.0144 \times ADT^{0.37} \times Segment\ Length^{0.83}$	1.194

Table 31. SPFs for Rural Four-Lane Divided Roadways.

Source	Range	RwD SPF	Inverse Dispersion Parameter
KABCO Crashes			
Texas	0.1 < Segment Length ≤ 0.77	$RwD\ KABCO\ Crashes = 0.0094 \times ADT^{0.61} \times Segment\ Length^{0.72}$	1.318
	0.77 ≤ Segment Length ≤ 2.0 ADT ≤ 13,000	$RwD\ KABCO\ Crashes = 0.0036 \times ADT^{0.72} \times Segment\ Length^{0.82}$	2.548
	0.77 ≤ Segment Length ≤ 2.0 ADT > 13,000	$RwD\ KABCO\ Crashes = 0.0025 \times ADT^{0.77} \times Segment\ Length^{0.87}$	2.100
KAB Crashes			
Texas	0.1 < Segment Length ≤ 0.77	$RwD\ KAB\ Crashes = 0.0118 \times ADT^{0.44} \times Segment\ Length^{0.78}$	0.969
	0.77 ≤ Segment Length ≤ 2.0	$RwD\ KAB\ Crashes = 0.0012 \times ADT^{0.69} \times Segment\ Length^{1.05}$	2.077

Safety Measures of Individual Project-Based Work Code

Using the SPFs above and the data collected from HSIP projects (as described in Chapter 3), the research team performed 340 individual project evaluations—two evaluations for each individual project (total RwD crashes, and KAB RwD crashes). The performance measure was the EB index of effectiveness. The calculation of this safety measure is described in the prior section. It is worth noting that in some cases, the safety measure index cannot be computed. For example, a safety measure cannot be calculated using the naïve method when the sum of crashes in the before period or the sum of crashes in the after period is zero. Although the EB method can be applied if the sum of crashes in the before period is zero, it is still undetermined when no crashes are recorded in the after period (which was the case for some projects in this study). Additionally, there were several projects for which there was no applicable SPF available (e.g., lower functional classes); thus, the safety index of effectiveness could not be calculated.

For example, Table 32 lists the safety measures of "Safety Treat Fixed Objects" (WC 209) for rural two-lane undivided roadways. Of the 166 segments on rural two-lane roadways, 26 sites had safety data that suggest positive safety impacts in KABCO crashes. Of the remaining segments, 49 sites had one or more crashes in the before period and zero crashes in the after period. Additionally, the safety measure could not be determined for 31 of the segments when considering KABCO crashes because no crashes were recorded in either the before or the after period.

Table 32. Safety Data Characteristics for WC 209 Projects at Rural Two-Lane Undivided Roadways.

<table>
<tr><td colspan="3" rowspan="2">Safety Measures of Individual Project</td><td colspan="2">RwD KABCO</td><td colspan="2">RwD KAB</td></tr>
<tr><td>Naïve rate comparison</td><td>EB</td><td>Naïve rate comparison</td><td>EB</td></tr>
<tr><td colspan="2">$Measure < 1.0$</td><td>Effective</td><td>26</td><td>22</td><td>7</td><td>1</td></tr>
<tr><td colspan="2">$Measure > 1.0$</td><td>Not effective</td><td>32</td><td>36</td><td>10</td><td>16</td></tr>
<tr><td rowspan="3">$Safety\ Measure$ determination</td><td>Crash rate before > 0
Crash rate after = 0</td><td>Potentially effective</td><td>49</td><td>49</td><td>48</td><td>48</td></tr>
<tr><td>Crash rate before = 0
Crash rate after > 0</td><td>Potentially not effective</td><td>28</td><td>28</td><td>26</td><td>26</td></tr>
<tr><td>Crash rate before = 0
Crash rate after = 0</td><td>Cannot be determined</td><td>31</td><td>31</td><td>75</td><td>75</td></tr>
</table>

Similar tables showing the number of projects and amount of data for other WCs are listed in Appendix IV (Table 35 through Table 46).

Effectiveness of Work Code for Group of Segments

One of the main objectives of this study was to determine the safety effectiveness of work codes for groups of segments to provide an overall understanding and establish a robust estimate of the expected performance. This is also known as combined treatments or multiple treatments. Table 33 lists historical crash information by the WCs. It is important to note that the acquired data have some limitations. For example, the research team needed to discard some segments due to the absence of installation dates.

Table 33. Before-After Historical Crash Information.

WC	Countermeasure Name	Facility Type	Number of Segments	Before Months (Avg.)	After Months (Avg.)	Before Crash	After Crash	Before Crash/ month	After Crash/ month
206	Improve Guardrail to Design Standards	Rural Four-Lane Undivided	4	56	44	6	14	0.027	0.079
209	Safety Treat Fixed Objects	Rural Two-Lane Undivided	166	42	31	320	235	0.046	0.046
209	Safety Treat Fixed Objects	Rural Four-Lane Undivided	18	45	18	47	23	0.059	0.072
209	Safety Treat Fixed Objects	Rural Four-Lane Divided	12	56	20	17	13	0.025	0.054
504	Construct Paved Shoulders (1–4 ft)	Rural Two-Lane Undivided	4	37	37	12	4	0.082	0.027
504	Construct Paved Shoulders (1–4 ft)	Rural Four-Lane Undivided	1	22	51	5	36	0.232	0.707
532	Texturize Shoulders (Rolled in Or Milled in/Profile Pavement Markers)	Rural Two-Lane Undivided	17	54	28	44	22	0.048	0.046
532	Texturize Shoulders (Rolled in Or Milled in/Profile Pavement Markers)	Rural Four-Lane Undivided	15	53	46	68	56	0.086	0.081
532	Texturize Shoulders (Rolled in Or Milled in/Profile Pavement Markers)	Rural Four-Lane Divided	5	58	7	24	2	0.083	0.058
536	Widen Paved Shoulders to > 5 ft	Rural Four-Lane Divided	13	57	24	6	1	0.008	0.003
206, 209	Improve Guardrail to Design Standards, Safety Treat Fixed Objects	Rural Two-Lane Undivided	60	45	32	128	70	0.047	0.037
206, 209	Improve Guardrail to Design Standards, Safety Treat Fixed Objects	Rural Four-Lane Undivided	14	15	56	17	54	0.083	0.069

Table 34 lists the safety effectiveness of the WCs by the facility types. This study developed 24 CMFs for six WCs associated with different roadway facility and crash severity types. For KABCO crashes, 10 CMFs show positive safety effectiveness (CMF < 1).

Table 34. Effectiveness of Work Codes by Facility Type.

WC	Counter-measure Name	Facility Type	No. of Segments	KABCO RwD CMF	SE of CMF	90% CI	90% Sign?	KAB RwD CMF	SE of CMF	90% CI	90% Sign?
206	Improve Guardrail to Design Standards	Rural Four-Lane Undivided	4	1.007	0.347	0.327–1.686	No	2.067	1.193	0.000–4.405	No
209	Safety Treat Fixed Objects	Rural Two-Lane Undivided	166	0.908	0.073	0.765–1.051	No	1.131	0.157	0.824–1.438	No
209	Safety Treat Fixed Objects	Rural Four-Lane Undivided	18	0.481[1]	0.121	0.244–0.717	Yes	1.671	0.679	0.341–3.002	No
209	Safety Treat Fixed Objects	Rural Four-Lane Divided	12	0.235[2]	0.068	0.102–0.367	Yes	0.146	0.104	0.000–0.351	Yes
504	Construct Paved Shoulders (1–4 ft)	Rural Two-Lane Undivided	4	0.853	0.496	0.000–1.824	No	0.674	0.674	0.000–1.995	No
504	Construct Paved Shoulders (1–4 ft)	Rural Four-Lane Undivided	1	3.406	2.176	0.000–7.671	No	2.186	1.756	0.000–5.629	No
532	Texturize Shoulders (Rolled In or Milled In/Profile Markers)	Rural Two-Lane Undivided	17	0.550	0.132	0.292–0.809	Yes	0.232	0.166	0.000–0.557	Yes
532	Texturize Shoulders (Rolled In or Milled In/Profile Markers)	Rural Four-Lane Undivided	15	0.909	0.153	0.610–1.208	No	1.359	0.433	0.511–2.207	No
532	Texturize Shoulders (Rolled In or Milled In/Profile Markers)	Rural Four-Lane Divided	5	0.209	0.150	0.000–0.504	Yes	—	—	—	—
536	Widen Paved Shoulders to > 5 ft	Rural Four-Lane Divided	13	0.010	0.010	0.000–0.030	Yes	—	—	—	—
206, 209	Improve Guardrail to Design Standards, Safety Treat Fixed Objects	Rural Two-Lane Undivided	60	0.540	0.072	0.399–0.681	Yes	0.909	0.197	0.522–1.295	No
206, 209	Improve Guardrail to Design Standards, Safety Treat Fixed Objects	Rural Four-Lane Undivided	14	0.675	0.110	0.459–0.891	Yes	1.358	0.404	0.566–2.149	No

[1] Blue cells indicate that the effectiveness of the WC is within the 90% confidence interval (CI) sign.
[2] Gray cells indicate that the effectiveness of the WC is within the 90% CI sign. However, the results need to be carefully interpreted due to the sample size issue.

However, for the 90 percent confidence interval (CI) values in Table 34, seven CMFs (blue and gray cells) show suggestive statistical evidence of a positive safety effectiveness. For KAB crashes, only four CMFs showed positive safety effectiveness (with two CMFs showing safety effectiveness within 90 percent CI values). Additionally, three CMFs were identified with surprisingly large and optimistic safety effects. The reason for these dubious results is their basis on low sample sizes (either by number of sites or number of crashes), and therefore care is advised when using those estimates.

SUMMARY OF WORK CODE UPDATE WORK

The research team collected safety data from multiple projects that implemented countermeasure treatments for roadway departure crashes. A large database of projects for evaluation was assembled, including multiple TxDOT data sources. Texas-specific roadway departure SPFs developed by another study were leveraged in this effort. The EB method was used to estimate the safety effectiveness of each project. All results were combined (as shown in Table 34) to assess the overall average safety effectiveness of each countermeasure.

In summary, results were found to be intuitive for the most part. Only a few of the 24 CMFs estimated were found to be statistically significant (nine out of 24). Regardless, four of the statistically significant CMFs yielded overly positive estimates due to being based on a small dataset. More details and discussion on these results are provided in Chapter 6.

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

This chapter provides conclusions and recommendations for the work described in this report. Two sections are provided, one for each of the efforts in this study: (a) a systemic approach for roadway departure crashes in Texas; and (2) recommended updates for select work codes in Texas.

SYSTEMIC APPROACH FOR ROADWAY DEPARTURE CRASH MITIGATION IN TEXAS

Roadway departures account for the majority of fatal crashes in rural areas. This research study developed a systemic approach method for prioritizing countermeasures at sites with increased risk of roadway departure, guardrail, and fixed-object crashes. The approach is based on a probability sample of Texas roadways and produced a set of risk factors for each crash type just described.

The method for roadway departure crashes included four variables: posted speed limit, horizontal curve density, shoulder width, and lane width. This method is applicable to identify two-lane rural highway locations with potential for any treatments installed to reduce roadway departure crashes. Some of the treatments that an analyst could propose after applying this method include rumble stripes/strips, raised pavement markers, profile markings, curve warning signs, delineators, and widened shoulders.

The primary purpose of a guardrail is to prevent a vehicle from striking a fixed object or traveling a terrain feature that is considered more dangerous than hitting the guardrail barrier when it inadvertently leaves the road. Crashes involving hitting the guardrails in rural areas are rare and random. It is often not possible to prioritize the sites based on crash history. The research team developed a systemic method for prioritizing candidate sites for guardrail improvements. The method included two variables: posted speed limit and guardrail offset. The analysis results showed that the likelihood of these crash types increases on higher-speed roadways. Also, the chance of hitting the guardrails decreases if they are located farther from the paved surface. The treatments where this method can be used include Modernize Bridge Rail and Approach Guardrail (TxDOT WC 205), Improve Guardrail to Design Standards (TxDOT WC 206), Install Guardrail or Roadside Barriers (TxDOT WC 207), and Safety Treat Fixed Objects (TxDOT WC 209).

A significant percent of motor vehicle crash deaths result from a vehicle leaving the roadway and hitting a fixed object alongside the road. Trees and utility poles are the most common objects struck. Safety-treating roadside trees and utility poles helps in reducing fatal crashes. The research team developed a systemic method for prioritizing sites for safety-treating fixed objects on the roadside. The method included five variables: posted speed limit, horizontal curve density, shoulder width, lane width, and horizontal clearance to fixed objects. The treatments where this method can be used include Install Guardrail or Roadside Barriers (TxDOT WC 207), Safety Treat Fixed Objects (TxDOT WC 209), and Modernize Facility to Design Standards (TxDOT WC 501).

Applicability to District-Specific Conditions

During discussions with TxDOT, questions about the representativeness of the proposed systemic methods at the district level were posed. The research team acknowledges that crash experiences and crash types would differ by region of the state, mostly due to different areas having distinct driving contexts and perhaps driving populations of different idiosyncrasies. However, the systemic methods proposed in this report are based on the risk factors present at specific segments. The information (risk factors and point weights) was extracted from multiple sites across the state and is not conditional to a specific region of the state. Since the risk factors are geometric or design features, it is anticipated that two segments with the same features (or risk factors) but in different districts would have comparable propensities to experience roadway departure crashes. A higher proportion of tree collisions at a densely forested district compared to a lower proportion at a district with few trees and curves is explained by how many miles of road the first district has with short lateral clearance (one of the risk factors identified in this research) and horizontal curve density (another risk factor), compared to the second district.

The screening methods hereby provided should be useful in either of these two scenarios because the prioritization criteria are the road features. The systemic approach would be applied to a larger pool of candidate sites for fixed-object treatments in the first hypothetical district, and thus the recommended countermeasures would be sensitive to the district-specific needs to reduce roadway departure crashes.

Recommendations for Future Work

Based on the rationale explained in the prior section, the research team believes that the methods as provided are applicable to different districts, though two future steps are recommended in that regard.

- A validation case study, perhaps from one or two volunteer districts.
- Contingent to successful validation, an implementation project to disseminate the methodology and develop an appropriate self-calculating tool, such as a smart spreadsheet.

The following are additional recommendations for future work.

- For this effort, the research team obtained the side-slope information for all the segments. Although this variable is used in the analysis, it was not found to be significant. The main reason is that the crashes are influenced by the side slope as well as how far the ditch is located from the traveled way. The latter could not be obtained in this study due to lack of readily available tools. It is recommended to obtain this variable and develop a systemic methodology for rollover crashes.
- The chance of hitting a guardrail increases if it is located on the horizontal curve. Although information about horizontal curves was collected in this study, a separate database is needed that includes guardrails on straight sections or horizontal curves. Future work should look into the influence of horizontal curves on guardrails.

UPDATES TO SELECT ROADWAY DEPARTURE WORK CODES

Regarding the effort to update select roadway departure WCs, the key findings are summarized next.

- The research team developed 24 CMFs for six WCs based on roadway facility and crash severity types. Around 38 percent of the CMFs obtained from the EB method were statistically significant at the 95 percent confidence level (rows highlighted in blue or gray in Table 34).
- The safety effectiveness of the WCs was developed separately for facility type and crash severity levels. When interpreting the safety effectiveness and assessing the lack of statistical significance in many cases (CMF, and range of CMF values), it is important to keep in mind the data limitations faced in some of the evaluations. For example, a very limited number of segments was available for the following WCs:
    - Improve Guardrail to Design Standards (WC 206) for Rural Multi-Lane Undivided: only four segments were available.
    - Construct Paved Shoulders (1–4 ft) (WC 504) for Rural Two-Lane Undivided: only four segments were available.
    - Construct Paved Shoulders (1–4 ft) (WC 504) for Rural Multi-Lane Undivided: only one segment was available.
    - Texturize Shoulders (Rolled In or Milled In/Profile Pavement Markers) (WC 532) for Rural Multi-Lane Divided: only five segments were available.
- In general, the findings of the current study show that CMFs of the WCs for KABCO crashes tend to be lower (i.e., indicating larger crash reductions) than CMFs for KAB crashes. However, two WCs (209 for rural multi-lane divided roadways and 532 for rural two-lane undivided roadways) show lower CMFs for KAB crashes.
- In this study, surprisingly, it was not found that WC 206 (Improve Guardrail to Design Standards) shows a positive safety effectiveness. Lower sample size is the presumed cause of this finding. A larger study in the literature (Cafiso et al., 2017) showed positive safety effectiveness (CMF = 0.67, SE = 0.22) for Improve Guardrail.
- WC 209 (Safety Treat Fixed Objects) shows positive safety effectiveness regarding KABCO crashes for four-lane divided and undivided facilities. A positive effectiveness was found for four-lane divided only with respect to KAB crashes. Findings from other studies (Hovey & Chowdhury, 2005; Ogle et al., 2009) are also in line with the findings of this study.
- For the combination work code (WC 206, 209), the safety effectiveness is generally positive (with the exception of rural four-lane undivided roadways for KAB crashes, where the analysis did not provide evidence of improvement).
- WC 532 (Texturize Shoulders [Rolled In or Milled In/Profile Pavement Markers]) shows positive safety effectiveness at rural two-lane undivided highways. The analysis did not offer evidence of an improvement at rural four-lane undivided roadways.

Caution is advised when interpreting some of the CMFs developed because they indicate very optimistic estimates of safety effectiveness. For example, WC 536 (Widen Paved Shoulders to > 5 ft) for rural multi-lane divided roadways was found to have a CMF of 0.010 (90 percent CI: 0.000–0.030), which is extremely optimistic. It should be noted that this CMF was developed from 13 segments with an average of 57 months of before crash data and 24 months of after crash data, but one crash was recorded in the 24 after-period months. This explains the extremely low CMF value. However, having long periods with no crashes could also be expected when there is a large amount of Poisson over-dispersion. Additionally, past research has found safety reductions for this treatment, but those results are significantly less optimistic than the result found in this research. Similar situations were found for the evaluations of WC 209 and WC 532 on rural four-lane divided highways. For this reason, the research team does not recommend adopting the updated CMFs for WC 209, WC 532, and WC 536 for rural four-lane divided highways.

The research team recognizes that the limited number of WCs recommended for updating resulted from insufficient data for the methodology elected for this effort. Although the EB method is considered the gold standard for before-after safety observational studies, its limitation to handle zero crash occurrences in the after period proved significant in this effort. The research team recommends further work analyzing the database developed for this effort using more advanced analytical methods to allow uncovering information from the cases that could not be analyzed using the EB method. The research team believes that obtaining such data-driven information is beneficial, even if it is from an alternative analytical method, especially if it is believed that some current WC values may not be based on a data-driven approach.

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APPENDICES

APPENDIX I: DATA COLLECTION PROTOCOL FOR AERIAL VIEW AND STREET VIEW

Problem flag: populate the field with these codes:

1 = no problems found.
2 = segment shorter than 0.01 mi.
3 = Google Earth photo quality is poor, or the street view is not available.
4 = segment under construction (Google Earth photo shows construction at some point during 2013–2017). Use the historical imagery view in Google Earth and briefly review all available photos during the years 2013–2017 to determine if construction occurs on a given segment.
5 = segment has passing lanes or two-way left-turn lane.
6 = if the segment has more than three curves.
7 = discrepancy between the street view and the aerial view.

- lane_width (feet): Average lane width for the traveled way. This width is determined by first measuring the surface_width (i.e., excluding shoulders), and then this width is divided by 2.
- l_shld_width (feet): Enter the width of the shoulder that is on the left when the vehicle is moving in the increasing milepost direction. Measure to the edge of pavement (exclude gravel).
- r_shld_width (feet): Enter the width of the shoulder that is on the right when vehicle is moving in the increasing milepost direction. Measure to the edge of pavement (exclude gravel).
- nbr_curves: Count of curves on the segment. Count includes any curve that is wholly or partially on the segment. A curve can be identified by drawing a straight construction line along a pavement marking. A curve begins where the marking diverges from the construction line. This technique is illustrated in Figure 33. Figure 33 shows one full segment. The left side of the segment includes an entire curve. The right side includes a part of a curve. The value for nbr_curves is 2.

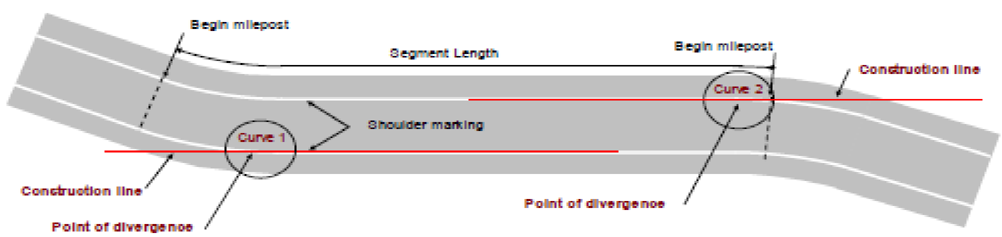

Figure 33. Horizontal Curve Location Technique.

- Clear_zone_width (feet): Measure the width of the clear zone along the segment. This measurement is specific to vertical objects in the roadside zone. It does not consider side slope or roadside barrier/guardrail. It is measured from the outside edge of highway traveled way to the nearest continuous line of vertical objects that are roughly parallel to the highway centerline and likely on the edge of the right of way. This line is typically indicated as a tree line, fence line, or utility poles, as shown in Figure 34. If the measured width exceeds 50 ft, then enter 50 ft.

Occasionally, a vertical object of sufficient size to represent a hazard is found in the clear zone but it is not part of a continuous line of objects. A solitary tree is the most common example of this situation. The clear zone is not measured to this lone object.

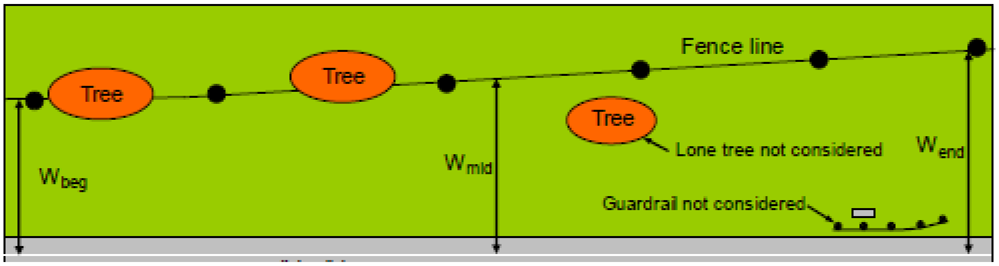

Figure 34. Clear Zone Width.

- Shld_rumble: Street View is used for this activity. This determination is made by checking both roadbeds. Use Street View and move along the increasing milepost direction. While using Street View, also check for centerline rumble strip and presence of guardrail. Enter 1 if rumble strip is present, 0 otherwise. Figure 35 shows the road segment with both shoulder and centerline rumble strips.

Figure 35. Centerline and Shoulder Rumble Strips.

- Center_rumble: Street View is used for this activity. Enter 1 if rumble strip is present, 0 otherwise.
- Speed_limit: Use Street View and move along the increasing milepost direction. Stop when you see a posted speed limit sign and enter the value. If you cannot find a speed limit sign, then check in the decreasing direction too. If you cannot find a speed limit sign in either direction, then move beyond the segment until you find one.
- Chevrons: Street View is used for this activity. Enter 2 if present in both directions, 1 if present in one direction only, and 0 otherwise. Figure 36 shows the road segment with chevrons.

Figure 36. Presence of Chevrons.

- Delineators: Street View is used for this activity. Enter 2 if present in both directions, 1 if present in one direction only, and 0 otherwise. Figure 37 shows the road segment with delineators.

Figure 37. Presence of Delineators.

- Driveways: In most cases, the aerial view is sufficient, but the street view may need to be used for this activity. Enter the total number of driveways along the segment.
- Minor_int: In most cases, the aerial view is sufficient, but the street view may need to be used for this activity. Enter the total number of minor intersections along the segment. If there are two intersections within 200 ft of driveway length, consider as only 1.
- Guardrail: Street view is used to confirm the presence of the guardrail. Using the measuring tool in Google Earth, measure the total length of the guardrail in both directions and enter the value in feet. Enter 0 if there is no guardrail.
- Guardrail_ht: When guardrail is present, measure the height of the actual guardrail face and enter the value in feet.
- GR_dist: Measure the distance from the paved shoulder to the GR and enter the value in feet.

- End_terminal_width: When guardrail is present, measure the width of the end terminal and enter the value in feet.
- End_terminal_ht: When guardrail is present, measure the height of the end terminal and enter the value in feet.
- End_terminal_type: When guardrail is present, enter the type of end terminal, such as rectangular, rounded, square, angled into ground, etc.
- num_poles: Count of total poles in both directions on the segment. If there is a guardrail acting as a fixed object, poles in that direction are not to be counted.
- Pole_dist: Average distance from the paved shoulder to the pole. Measure the distance to each pole and then take the average. Enter the value in feet.
- num_lone_trees: Count of total lone trees in both directions on the segment.
- Lone_Tree_dist: Average distance from the paved shoulder to the lone tree on the roadside. Measure the distance to each tree in the segment and then take the average. Enter the value in feet.
- side_slope: Measure the side slope in both directions and enter the MAXIMUM value.
- vert_grade: Measure the grade of all vertical curves in the segment and enter the MAXIMUM value.

APPENDIX II: DATA COLLECTION PROTOCOL FOR PLACING PINS ON HORIZONTAL CURVES

1. Right click on "Temporary Places" in Places, click Add, click Folder, and in Name enter in the following format: hwy_seg_num (e.g., for the seg_num=1, the name will be FM0922_1), then click OK. The folder is created for just those segments that have curves on them.
2. Add markers for the segment beginning DFO and ending DFO to the folder. If the curve is far away from the beginning DFO, enter a few markers and name them from 1, 2, ... until a few feet before the curve.
3. Put markers along the curve. They MUST be listed in Places in the order below (also in the increasing direction of the mile point)

Label	Need	Description
s1 i	Required	Locate on tangent at least 150 ft before the curve.
s2 i	Required	Locate on tangent at least 100 ft after s1 and before the curve.
m1 i	Required	Locate on curve at least 50 ft after start of the curve.
m2 i	Required	Locate at least 50 ft and not more than 300 ft from m1.
:		Points m1, m2, ... mN are located along the curve with shorter spacing (maximum 300 ft). At least 3 points are needed.
mN i	Required	Locate at least 50 ft from last point.
e1 i	Required	Locate on tangent at least 50 ft after the curve.
e2 i	Required	Locate on tangent at least 100 ft from e1 in the direction away from the curve (i.e., at least 150 ft from the curve).
The letter "i" in each variable name is the curve number (i = 1, 2, 3, etc.). The total number of curves should be equal to the "number of curves" populated earlier.		
If there are two or more curves in the file, then enter a unique curve number for all markers associated with a curve.		

1. Name each marker using the label in the list above. If the curve is far away from the beginning DFO enter a few markers and name them from 1, 2, etc.
2. After adding the markers, right click on the file/folder created in Step 1 and go to next step.
3. Click Save Place As (use default file name, do not change it at this point). Save the file as type .kml (select at bottom of the active file-save window) in this location (…\Tasks 0-6991 (Raul Avelar)\Task 3\Data Collection from P Sample\Horizontal curves).

Note:

1. Put markers on the centerline.
2. If the beginning DFO is on a tangent section, then the first pin in the folder MUST be the beginning DFO (and its name should be the actual DFO, e.g., 10.05).
3. If the beginning DFO is on a curve, then measure to a point that is 250 ft away from the start of curve. Now, the first pin in the folder must be this point, and the name will be its actual DFO. If the beginning DFO is 0, then this point will have a negative value.

APPENDIX III: DOWNLOADING IMAGES FROM GOOGLE STREET VIEW

1. Open “start_coords_6991” and “Sections_sampled_06991” in Google Earth to view the identified segments. (The file “start_coords_6991” represents the starting point of each segment and is represented by the object ID shown in column C of the spreadsheet. The second file, “Sections_sampled_06991,” shows the full segment.)
2. Using Object ID, identify the segment for which the analysis needs to be conducted. Use the street view mode in Google Earth Pro to view the on-ground characteristics of the segment.
3. Check the length of the segment. If the length of the segment < 0.5 mi, one single image toward the middle of the segment should be enough. If the segment length is between 0.51 mi and < 1 mi, collect two images (one in the beginning and one toward the end). If segment is > 1 mi, collect three images (beginning, middle, and end).
4. Using the street view mode in Google Earth Pro, locate the marker where the analysis needs to be conducted. Click on File>View in Google Maps. (Alternatively, the image location can be copied from the edit tab in the menu option.) This should open Google Maps online using your default browser.
5. Using the street view in Google Maps, enter the street view mode. Align the direction of the segment to make sure that the segment is roughly in the middle of the viewing screen.
6. In the address bar, change the “field of view” (y) URL parameter to 60 y. Also change the “tilt” (t) to 85 t. Click “enter” to reload the street view. This should reload the same street view with specified parameters.
7. Take a screenshot of the street view using the PrintScn option on the keyboard. If working on two monitor screens, it might be necessary to use Alt+PrtScn.
8. In a separate Word file, paste the screenshot of the street view. Be sure to label the image according to the object ID and image ID.
9. Copy the address of the image along with the image in the word file.
10. Right click on the image, select Save as Picture and save the file as a JPEG in the respective JPG folder in Syncplicity.

Notes:

1. Check for guardrails in the segment. If guardrail is present, make sure to collect at least one image from each guardrail section for analysis.
2. Once the street view is reloaded with specified parameters, do not click anywhere on the maps or else the parameters might change.

The number of lone trees, poles, and cluster of trees within 50 ft from the edge of the lane are identified. The length of any three clusters of trees is measured from the aerial view, and the number of trees present in the cluster of trees are counted from the street view. Please note that the number of trees are counted if the cluster length is less than 200 ft. The average length of the cluster of trees and the average number of trees in each segment are then calculated.

In order to calculate the effective length of road which is under influence of a major collision with the objects on the sides of the road, the deflection angle and maximum of all the lengths of the clusters of trees (in cases of clusters of trees) are needed. In order to maximize the effective length, the deflection angle is considered as 10 degrees from the entering edge of the object and as 25 degrees from the leaving edge of the object. The effective length is calculated as follows:

$$Effective\ length = Maximum\ length\ of\ object + \ Average\ distance\ of\ pole\ (or\ lone\ tree) * [Cot10 - \ Cot25]$$

In case of a pole and lone trees, the entering edge and the leaving edge is at the same point, so the maximum length of object = 0. For a cluster of trees, the maximum length of object is equal to the maximum length of the cluster of trees. The separate effective lengths are calculated for cluster lengths less than and greater than 200 ft.

APPENDIX IV: ADDITIONAL SUMMARIES OF SAFETY DATA CHARACTERISTICS BY WORK CODE

Table 35. Safety Data Characteristics for WC 209 Projects at Rural Four-Lane Undivided.

Safety Measures of Individual Project			RwD KABCO		RwD KAB	
			Naïve rate comparison	EB	Naïve rate comparison	EB
$Safety\ Measure < 1.0$		Effective	5	5	2	0
$Safety\ Measure > 1.0$		Not effective	1	1	0	2
$Safety\ Measure$ determination	Crash rate before > 0 Crash rate after = 0	Potentially effective	11	11	10	10
	Crash rate before = 0 Crash rate after > 0	Potentially not effective	1	1	2	2
	Crash rate before = 0 Crash rate after = 0	Cannot be determined	0	0	4	4

Table 36. Safety Data Characteristics for WC 209 Projects at Rural Four-Lane Divided.

Safety Measures of Individual Project			RwD KABCO		RwD KAB	
			Naïve rate comparison	EB	Naïve rate comparison	EB
$Safety\ Measure$ determination < 1.0		Effective	1	3	0	1
$Safety\ Measure$ determination > 1.0		Not effective	2	0	1	0
$Safety\ Measure$ determination	Crash rate before > 0 Crash rate after = 0	Potentially effective	1	1	3	3
	Crash rate before = 0 Crash rate after > 0	Potentially not effective	4	4	1	1
	Crash rate before = 0 Crash rate after = 0	Cannot be determined	4	4	7	7

Table 37. Safety Data Characteristics for WC 206 Projects at Rural Two-Lane Undivided.

Safety Measures of Individual Project			RwD KABCO		RwD KAB	
			Naïve rate comparison	EB	Naïve rate comparison	EB
$Safety\ Measure < 1.0$		Effective	0	0	0	0
$Safety\ Measure > 1.0$		Not effective	0	0	0	0
$Safety\ Measure$ determination	Crash rate before > 0 Crash rate after = 0	Potentially effective	2	2	2	2
	Crash rate before = 0 Crash rate after > 0	Potentially not effective	0	0	0	0
	Crash rate before = 0 Crash rate after = 0	Cannot be determined	9	9	9	9

Table 38. Safety Data Characteristics for WC 206 Projects at Rural Four-Lane Undivided.

Safety Measures of Individual Project			RwD KABCO		RwD KAB	
			Naïve rate comparison	EB	Naïve rate comparison	EB
$Safety\ Measure < 1.0$		Effective	0	2	0	0
$Safety\ Measure > 1.0$		Not effective	3	1	0	0
$Safety\ Measure$ determination	Crash rate before > 0 Crash rate after = 0	Potentially effective	1	1	1	1
	Crash rate before = 0 Crash rate after > 0	Potentially not effective	0	0	3	3
	Crash rate before = 0 Crash rate after = 0	Cannot be determined	0	0	0	0

Table 39. Safety Data Characteristics for WC 504 Projects at Rural Two-Lane Undivided.

Safety Measures of Individual Project			RwD KABCO		RwD KAB	
			Naïve rate comparison	EB	Naïve rate comparison	EB
$Safety\ Measure < 1.0$		Effective	1	1	1	0
$Safety\ Measure > 1.0$		Not effective	0	0	0	1
$Safety\ Measure$ determination	Crash rate before > 0 Crash rate after = 0	Potentially effective	1	1	1	1
	Crash rate before = 0 Crash rate after > 0	Potentially not effective	1	1	0	0
	Crash rate before = 0 Crash rate after = 0	Cannot be determined	1	1	2	2

Table 40. Safety Data Characteristics for WC 504 Projects at Rural Four-Lane Undivided.

Safety Measures of Individual Project			RwD KABCO		RwD KAB	
			Naïve rate comparison	EB	Naïve rate comparison	EB
$Safety\ Measure < 1.0$		Effective	0	0	0	0
$Safety\ Measure > 1.0$		Not effective	1	1	1	1
$Safety\ Measure$ determination	Crash rate before > 0 Crash rate after = 0	Potentially effective	0	0	0	0
	Crash rate before = 0 Crash rate after > 0	Potentially not effective	0	0	0	0
	Crash rate before = 0 Crash rate after = 0	Cannot be determined	0	0	0	0

Table 41. Safety Data Characteristics for WC 532 Projects at Rural Two-Lane Undivided.

Safety Measures of Individual Project			RwD KABCO		RwD KAB	
			Naïve rate comparison	EB	Naïve rate comparison	EB
$Safety\ Measure < 1.0$		Effective	3	6	1	2
$Safety\ Measure > 1.0$		Not effective	5	2	1	0
$Safety\ Measure$ determination	Crash rate before > 0 Crash rate after = 0	Potentially effective	3	3	5	5
	Crash rate before = 0 Crash rate after > 0	Potentially not effective	1	1	0	0
	Crash rate before = 0 Crash rate after = 0	Cannot be determined	5	5	10	10

Table 42. Safety Data Characteristics for WC 532 Projects at Rural Four-Lane Undivided.

Safety Measures of Individual Project			RwD KABCO		RwD KAB	
			Naïve rate comparison	EB	Naïve rate comparison	EB
$Safety\ Measure < 1.0$		Effective	7	7	2	2
$Safety\ Measure > 1.0$		Not effective	4	4	3	3
$Safety\ Measure$ determination	Crash rate before > 0 Crash rate after = 0	Potentially effective	1	1	5	5
	Crash rate before = 0 Crash rate after > 0	Potentially not effective	2	2	2	2
	Crash rate before = 0 Crash rate after = 0	Cannot be determined	1	1	3	3

Table 43. Safety Data Characteristics for WC 532 Projects at Rural Four-Lane Divided.

Safety Measures of Individual Project			RwD KABCO		RwD KAB	
			Naïve rate comparison	EB	Naïve rate comparison	EB
$Safety\ Measure < 1.0$		Effective	1	2	0	0
$Safety\ Measure > 1.0$		Not effective	1	0	0	0
$Safety\ Measure$ determination	Crash rate before > 0 Crash rate after = 0	Potentially effective	2	2	3	3
	Crash rate before = 0 Crash rate after > 0	Potentially not effective	0	0	0	0
	Crash rate before = 0 Crash rate after = 0	Cannot be determined	1	1	2	2

Table 44. Safety Data Characteristics for WC 536 Projects at Rural Four-Lane Divided.

Safety Measures of Individual Project			RwD KABCO		RwD KAB	
			Naïve rate comparison	EB	Naïve rate comparison	EB
$Safety\ Measure < 1.0$		Effective	0	1	0	0
$Safety\ Measure > 1.0$		Not effective	1	0	0	0
$Safety\ Measure$ determination	Crash rate before > 0 Crash rate after = 0	Potentially effective	2	2	1	1
	Crash rate before = 0 Crash rate after > 0	Potentially not effective	0	0	0	0
	Crash rate before = 0 Crash rate after = 0	Cannot be determined	10	10	12	12

Table 45. Safety Data Characteristics for WC 206 and 209 Projects at Rural Two-Lane Undivided.

Safety Measures of Individual Project			RwD KABCO		RwD KAB	
			Naïve rate comparison	EB	Naïve rate comparison	EB
$Safety\ Measure < 1.0$		Effective	9	15	5	3
$Safety\ Measure > 1.0$		Not effective	14	8	5	7
$Safety\ Measure$ determination	Crash rate before > 0 Crash rate after = 0	Potentially effective	22	22	16	16
	Crash rate before = 0 Crash rate after > 0	Potentially not effective	7	7	7	7
	Crash rate before = 0 Crash rate after = 0	Cannot be determined	8	8	27	27

Table 46. Safety Data Characteristics for WC 206 and 209 Projects at Rural Four-Lane Undivided.

Safety Measures of Individual Project			RwD KABCO		RwD KAB	
			Naïve rate comparison	EB	Naïve rate comparison	EB
$Safety\ Measure < 1.0$		Effective	6	7	3	1
$Safety\ Measure > 1.0$		Not effective	4	3	0	2
$Safety\ Measure$ determination	Crash rate before > 0 Crash rate after = 0	Potentially effective	1	1	1	1
	Crash rate before = 0 Crash rate after > 0	Potentially not effective	3	3	5	5
	Crash rate before = 0 Crash rate after = 0	Cannot be determined	0	0	5	5