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Safety performance measures of bridges on county roads in Iowa

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DEDICATION

I would like to dedicate this thesis to my parents, Emmanuel Sr. and Comfort, my siblings, extended family, and friends. Without their support and encouragement, I could not have successfully completed this level of my education. Thank you all.

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DISCLAIMER

This document was prepared and written by the author to fulfill the requirements set forth by Iowa State University for the degree of Master of Science. The views expressed in this thesis are those of the author and do not reflect the viewpoint or policy of the Institute for Transportation (InTrans) at Iowa State University, the Bridge Engineering Center (BEC) at Iowa State University, the Iowa Department of Transportation, or Iowa State University (ISU).

ABSTRACT

Introduction: Roadway departure crashes, including those involving traffic barriers such as bridge rails and guardrails, tend to be frequent and severe in nature, specifically for roadways serving high traffic volumes at high speeds. In 2013, the National Highway Traffic Safety Administration (NHTSA) observed that, in the United States, collisions with fixed objects and non-collisions incidents account for only 18 percent of all reported crashes; however they result in 44 percent of all fatal crashes.

Methods: That said, this paper explored significant relationships between roadway elements, its surroundings, design, and condition characteristics using statistical analysis and regression modeling to better understand associative properties of roadway and bridge characteristics on the frequency and severity of crashes involving bridges and guardrails.

Results: The (crash) frequency results revealed that vehicular crashes involving bridges are very rare events. In the 10-year analysis period from 2004 to 2013, there were merely 862 single-vehicle bridge crashes reported in Iowa. Nonetheless, crashes involving bridges are more frequent on some bridges more than others. In conjunction with previous studies, bridges characterized to have increased traffic volumes and lengths are more susceptible to crashes. In general, bridges designed to old design standards or meeting substandard (superstructure, railing, or alignment) conditions are also more susceptible to crashes over time. Conversely, bridges characterized to have relatively wider widths than their travel ways, and bridges on paved roadways are less susceptible to crashes.

The (occupant injury) severity results revealed that collisions involving bridges tend to be more severe for unprotected vehicle occupants. Also, concurrent with past studies, driving under the influence of drug or alcohol increases the probability of more severe injuries. On the contrary, bridge crashes during wet, icy, snowy, or slushy road surface conditions however tend to be less severe.

Practical Applications: While the low quantities of bridge crashes on county roads may be indicative of bridge rails and guardrails serving their purpose, the findings of this study can be useful to local public agencies regarding guidance for bridge and barrier rail upgrade standards. Special consideration/emphasis may be placed on bridges possessing certain characteristics because they expect higher crashes, although a relatively small proportion of bridges may actually possess these characteristics.

That said, prescriptive guidelines for bridge rail use on county roads may not be necessary, given the rareness and randomness of crash events. However, other effective approaches may include the use of categorical thresholds for establishing practical requirements of when and how county engineers should upgrade bridges.

Keywords: low-volume roads, bridge guardrail safety, county road bridges, single-vehicle crashes, bridge crash frequency, bridge crash severity

CHAPTER 1 INTRODUCTION

This chapter briefly introduces design, traffic, and environmental concerns related to bridge safety within the notion of transportation engineering. Section 1.1 of this paper discusses the background and motivating issues of the study. Section 1.2 discusses the objectives of this study. Section 1.3 presents the thesis organization.

1.1 Background

Traffic barriers, which includes bridge components such as railings and guardrails, have been a fixture along America's roadways since there were carriageways. These type of fixed objects have always been present in our built environment and has flourished since steel became commercially available as a construction material in the mid-19th century. Nonetheless, according to the American Association for State Highway and Transportation Officials (AASHTO), roadside safety concept did not become a much discussed aspect in highway (road) design until the late 1960s, and it was the decade of the 1970s before roadside design was regularly incorporated into highway projects (AASHTO RDG 2011). Predominantly, found within close proximity (within the clear zone) of the traveled ways, railings and guardrails are designed to provide safety to users by shielding more hazardous objects and protecting errant vehicles from encroaching onto hazardous roadside objects when leaving their lane. Nevertheless, the addition of such roadside elements increases both the installation and maintenance costs of a roadway (or bridge) and adds another obstacle to the roadway likely to cause (or result in) a crash. Table 1 shows list of roadside objects most commonly struck in roadside fatal crashes, in descending order of frequency.

Table 1: Objects Most Commonly Struck in Fatal Crashes

1.	Trees
2.	Utility Poles
3.	Boulders
4.	Drainage Devices
5.	Embankments
6.	Guardrails

Source: (Stephens Jr. 2005)

Sometimes referred to as "off-pavement" design, roadside design is often defined as the design of the area outside the traveled way. Though the pros and cons of such design consideration have been questioned for many decades, it is perhaps their influence on traffic and safety that has been most controversial. According to the *Roadside Design Guide* (RDG), one commonly asked question revolves around whether spending resources on "off-the-pavement" improvements really are beneficial given the limited nature of infrastructure funds; even though, road departure crashes accounted for 54 percent (17,791 deaths) of all traffic fatalities in the United States as of 2014 (AASHTO RDG 2011). Furthermore, other positive (not excluding psychological and aesthetic) implications for the inclusion of roadside design elements to improve highway safety include the benefits associated with driver comfortability, roadway (utility) functionality, and beautification for road users. Roadway departure crashes involving these type of fixed objects tend to be more severe in nature, specifically for roadways serving high traffic volumes at high speeds. Table 2 shows crash statistics from the Fatality Analysis Reporting System (FARS) database prepared by the Insurance Institute of Highway Safety (IIHS) in 2003. As shown, though roadside fatalities occur more frequently at higher speeds, they can in fact occur at any speed.

Table 2: Deaths in Roadside Crashes, 2003

Speed Limit		Percent (%)
(miles per hour)	(kilometers per hour)	
30 mph or less	50 km/h or less	12%
35 to 40 mph	55 to 65 km/h	19%
45 to 50 mph	70 to 80 km/h	17%
55 mph or greater	90 km/h or greater	48%
No Limit or Unknown		4%
Total		100%

Source: (Stephens Jr. 2005)

The presence of roadside elements have always been disadvantageous to the comfort and safety of drivers as these objects can cause severe injuries during run-off-road collisions. Consequently, over the years, roadway design concepts (i.e. alignments and intersection designs) and vehicle designs have significantly improved through research, experience, and education to help reduce the total number of traffic fatalities each year. Nonetheless, limited effort has been done to improve the safety of fixed roadside elements such as bridge rails and guardrails, particularly along non-primary roadways. By reason of these (bridge) components close proximity to the edge of the traveled way, their presence in the clear zone constitutes major roadside safety hazards to road users. Figure 1 illustrates an example of an Iowa county roadway bridge with approach guardrail.

Figure 1: County Highway Bridge over the South Fork of the Iowa River in Hardin County, Iowa

Roadway departure crashes as defined by the Federal Highway Administration (FHWA) are crashes which occur after vehicles cross an edge line or centerline, or otherwise leave its travel lane. In 2013, the National Highway Traffic Safety Administration (NHTSA) observed that, in the United States, collisions with fixed objects and non-collisions incidents account for only 18 percent of all reported crashes; however they result in 44 percent of all fatal crashes (NHTSA 2013). More specifically, in accordance with the IIHS, the proportion of motor vehicle deaths involving roadside objects has remained relatively consistent over the years since 1979 (IIHS 2013). In many instances, these are errant single-vehicles colliding with roadside fixed objects such as trees, utility poles, sign posts, bridge piers, etc. Based on 2008 reported incidents, trees were observed to account for the majority (48%) of roadside crash fatalities, whilst utility poles and traffic barriers accounted for 12 and 8 percent, respectively (AASHTO RDG 2011). These figures imply that roadside environment has a very significant effect on run-off-road crashes. Thus, it is necessary to assess significant relationships between roadside objects (including bridge rails and guardrails) and roadway characteristics, to better improve the safety and countermeasures of “off-the-pavement” design. In so doing, that may help to reduce the severity of crashes and possibly frequencies of crashes involving roadside fixed objects.

Although crashes involving bridge rails or guardrails appear to be rare, random, and account for a small percentage of all vehicular crashes each year, these type of run-off-road crashes also account for a large portion of fatal crashes. Moreover, only 15 percent of these incidents occur on interstates and freeways, with an additional 49 percent occurring on other primary roadways according to the Insurance Institute for Highway Safety (IIHS 2013). There are many reasons why vehicles leave the traveled way and encroach onto the roadside, including:

- Driver fatigue or distractions,
- Excessive speeding,
- Driving under the influence of drugs or alcohol,
- Crash avoidance,
- Roadway environmental conditions such as ice, rain, or poor visibility,
- Vehicle component failure;

Regardless of reason, these crashes are typically more frequent along horizontal curvatures (60%) and on rural roads (41%); according to NHTSA, roadside crash fatality rate in rural areas (1.88) is approximately 2.5 times higher than the fatality rate in urban areas (0.77) (NHTSA 2013). Though, only 19 percent of the US population (less than 60 thousand people) lived in rural areas and (U.S. Census Bureau 2010). Efforts to reduce these driver errors are only effective when implemented appropriately; thus, it is important to shield or remove fixed objects or avoid placing them along roads in the first place, especially along roads where vehicles are more likely to leave the pavement. Within the concept of roadside safety, there are three (3) key components: vehicle safety, roadway safety, and driver behavior. Substantial advances has been made in each aspect due to several factors. Motor vehicles today are much safer than they have been in the past. Protected passenger compartments, padded interiors, occupant restraints, and airbags are examples of some features that have added to improve passenger and vehicle safety during impact situations. Whereas, roadways have been made safer through improvements in features such as horizontal and vertical alignments, intersection geometry, traversable roadsides, roadside barrier performance, and grade separations and interchanges, to name a few. Lastly, road users today are more educated about safe road/vehicle operations as evident by the increased use of occupant restraints and decreased rate of alcohol-impairment driving fatalities over time (NHTSA 2013). All these contributing factors have reduced the motor vehicle fatality rate over the years. Unfortunately, crashes involving roadside objects still account for far too great a portion of the total fatal highway crashes.

1.2 Research Objectives

Currently, most existing bridges (72%) on low-volume, county roads (LVRs) in the state of Iowa do not have bridge rails, guardrails, transitions, or guardrail ends that meet acceptable design standards. Thus, the primary goals of this research was to 1) evaluate the current practice of upgrading bridge rails on low volume roads in Iowa; 2) determine if the current practice/criteria is adequate; and 3) if not, propose new/different criteria to be used in the future for upgrades. Herewith, this paper explores the relationships between roadway elements including bridge rails and guardrails, its surroundings (i.e. traffic), condition, and design characteristics (i.e. dimensions) to better understand the associative properties of roadway and bridge characteristics on the frequency and severity of crashes involving bridges and guardrails. In provision of these goals, the total of 862 crashes occurring at/near 18,577 inventoried county road bridges in Iowa, a statistic representing less than 0.2% of the statewide reportable crashes over a 10-year period from 2004-2013, was used for this analysis. The original research objectives (hypotheses) of this paper are presented in twofold:

1. What characteristics (i.e. operation, design, condition) significantly influence the frequency of crashes involving bridges on county roadways?
2. What circumstantial attributes (i.e. person, vehicle, crash) significantly influence the likely outcome of occupant injury involving crashes with bridges on county roadways?

1.3 Thesis Organization

This thesis was organized into six chapters. The first chapter introduced a brief background and motivation in regards to the need to improve roadway roadside safety. The second chapter reviewed key policies, guidelines, and past studies relating to bridge and roadway safety which set the stage for this research. Besides, chapter two also summarized the past literatures relating to bridges in three perspectives, including design, safety operations, and sustainability. The third chapter described the data collection protocol and modifications that were used during the analyses. Additionally, this chapter also presented the procedures for the data quality assurance. Chapter four and chapter five presented the data methodology and analysis results of parameters considered in the crash frequency and crash severity models respectively. Additionally, statistical analysis and regression models were built to explain patterns within roadway design, operational, and conditional characteristics in both models. The last chapter (chapter six) summarized the findings, contributions to the state-of-arts, limitations, and recommendations for future research.

CHAPTER 2 LITERATURE REVIEW

This chapter summarizes key policies, guidelines, and literary findings relating to bridge-roadway design, safety operations, and sustainability. Section 2.1 of this paper summarizes recommended policies and guidelines relating to bridge design and construction. Section 2.2 reviews past literatures relating to the research goals. Section 2.3 summarizes relevant findings.

2.1 Policies and Guidelines

Modern highway design concepts essentially began in the 1940s; however, roadside safety design did not start until the 1970s (AASHTO RDG 2011). Today, many roadways built prior to 1970 have reached their useful designed lifespan and are prime candidates for reconstruction—an opportunity to update and improve their “off-the-pavement” designs. National- and state-level roadway design guidelines have been established to be used by states and local agencies as acceptable design standards/guidance and are regularly revised/refined over time. Released in 1967, the *Highway Design and Operational Practices Related to Highway Safety* was the first official report that focused attention on hazardous roadside elements and suggested appropriate treatments for them (AASHTO RDG 2011). The document was later revised and updated in 1974 with the introduction of roadside concepts by the American Association for State Highway Officials. In 1989, AASHTO published the first edition of the *Roadside Design Guide*. Through years of research and experience, the design guide has been modified over time to include sequential options for reducing crashes involving roadside obstacles. The following, in order of preference, are techniques suggested for reducing crashes and crash severity:

1. Remove the obstacle.
2. Redesign the obstacle so it can be safely traversed.
3. Relocate the obstacle to a point where it is less likely to be struck.
4. Reduce impact severity by using an appropriate breakaway device.
5. Shield the obstacle with a longitudinal traffic barrier designed for redirection or use a crash cushion.
6. Delineate the obstacle if the previous alternatives are not appropriate.

Often, the removal or relocation of such roadside obstacles may be impractical or unavoidable. Along roadways where the shortest lateral distance (i.e. horizontal clearance) to a roadside fixed objects is considered 'insufficient' or hazardous to user safety, some common cost-effective countermeasures include the installation of obstacle protective devices such as cable/traffic barriers, guardrails, or impact attenuators (crash cushions), the installation of "on-the-pavement" edge safety features such as shoulder rumble strips/stripes, or a combination of both to help errant vehicles recover after diverging from its traveled way before colliding with a roadside obstacle. In many cases, these counteragents may be appropriate and have been proven beneficial toward the reduction of the severity and possibly frequency of run-off-road crashes. Nonetheless, they do not completely explain the problem of serious injuries associated with roadway departure crashes involving roadside objects.

The AASHTO *Manual for Assessing Safety Hardware* (MASH) is the new state-of-the-practice for the crash testing of safety hardware devices for use on the National Highway System (NHS). Federal Highway Administration (FHWA) policy requires that all roadside appurtenances such as traffic barriers, barrier terminals and crash cushions, bridge (guard) railings, sign and light pole supports, and work zone hardware used on the NHS (or federally funded projects) shall meet full-scale crash performance criteria contained in the National Cooperative Highway Research Program (NCHRP) Report 350: *Recommended Procedures for the Safety Performance Evaluation of Highway Features* (NCHRP 1993), or AASHTO's MASH (AASHTO MASH 2009). Bridge railings are very important components of roadway safety systems and play an important role in preventing and mitigating crashes. Since its primary purpose is to prevent penetration, bridge railings must be strong enough to redirect an impacting vehicle.

AASHTO's MASH test criteria describe six levels of testing and crashworthiness ratings. It presents specific test level (TL) impact conditions at various speeds for conducting vehicle crash tests. The first three levels are based on impact speed, from 31 mph for TL-1 to 44 mph for TL-2, and 62 mph for TL- 3. Test Levels 1 – 3 are based on impacts from light vehicles such as passenger cars and light trucks. Whereas, Test Levels 4 – 6 contain additional tests for bridge railings designed to contain and redirect heavy vehicles such as buses and larger trucks. However, because of concerns with high speed conditions, test level 3 (TL-3), tested at 100 km/h (62 mph), devices are considered standard by many highway agencies (AASHTO MASH 2009). All new or replacement railing on NHS bridges must meet Test Level 3 crash-test criteria as a minimum. Table 3 shows the test matrix for traffic barrier systems. In many instances, TL-3 devices work for both TL-1 and TL-2 conditions as well as for high speed conditions. The FHWA also reviews test results and issues worthiness letters for each bridge rail that is tested according to the evaluation criteria. More details regarding the railing schematic and worthiness by type (W-Beam, Thrie Beam, concrete, timber, etc.) can be found in Table 19 of appendix A.

Table 3: Example of MASH Test Matrix for Traffic Barrier Systems

Test Level	Test Vehicle Designation and Type	Test Conditions		
		Vehicle Weight kg [lb]	Speed km/h [mph]	Angle Degree
1	1100C (Passenger Car)	1,100 [2,420]	50 [31]	25
	2270P (Pickup Truck)	2,270 [5,000]	50 [31]	25
2	1100C (Passenger Car)	1,100 [2,420]	70 [44]	25
	2270P (Pickup Truck)	2,270 [5,000]	70 [44]	25
3	1100C (Passenger Car)	1,100 [2,420]	100 [62]	25
	2270P (Pickup Truck)	2,270 [5,000]	100 [62]	25
4	1100C (Passenger Car)	1,100 [2,420]	100 [62]	25
	2270P (Pickup Truck)	2,270 [5,000]	100 [62]	25
	10000S (Single Unit Truck)	10,000 [22,000]	90 [56]	15
5	1100C (Passenger Car)	1,100 [2,420]	100 [62]	25
	2270P (Pickup Truck)	2,270 [5,000]	100 [62]	25
	36000V (Tractor/Van Trailer)	36,000 [79,300]	80 [50]	15
6	1100C (Passenger Car)	1,100 [2,420]	100 [62]	25
	2270P (Pickup Truck)	2,270 [5,000]	100 [62]	25
	36000T (Tractor/Tanker Trailer)	36,000 [79,300]	80 [50]	15

Source: (AASHTO MASH 2009)

From the perspective of bridge roadway design, right-of-ways, clear zones, and horizontal clearance are the three most important mechanisms to consider when assessing roadside safety improvements. Roadway right-of-way (R.O.W.) is simply parcel(s) of land reserved for public transportation purposes such as roads. They typically are cross-sections of a road, including all traveled lanes, shoulders, road signage, clear zones, and clearances etc. Whereas, AASHTO *Roadside Design Guide* defines clear zones as “the total roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles” (AASHTO RDG 2011). This area may consist of shoulders, recoverable and non-recoverable slopes, and/or clear run-out areas. The desired minimum width of a clear zone is dependent upon traffic volumes and speeds and on the roadside geometry. Simply stated, it is an unobstructed, relatively flat traversable roadside area beyond the edge of the traveled way designed to allow a driver to stop safely or regain control of a vehicle that leaves the traveled way. AASHTO RDG also provides guidance for evaluating the need for shielding steep slopes and roadside objects, including bridge rails. Tables in the design guide provide minimum clear zone requirements based on design speed, traffic volume (ADT), and slope. For example, for a roadway with a design speed of 55 mph, ADT less than 750, and fore slopes flatter than 1:4, a minimum clear zone of 12-18 feet is required. However, the design guide does not specifically address roadside design issues for very low-volume roads (i.e., ADT ≤ 400).

None of these are exactly the same as horizontal clearance; horizontal clearance can be defined as the lateral offset distance needed to provide safe passage for vehicular traffic from the edge of the traveled way, shoulder or other designated point, to a vertical roadside element (AASHTO Green Book 2004). Curbs, walls, barriers, piers, sign and signal supports, trees, street/parked cars, landscaping items, and utility poles are prime examples of the type of features that require horizontal clearance. Figure 2 and Figure 3 illustrate schematics of typical right-of-way, horizontal clearance, and clear zone sections in roadway design.

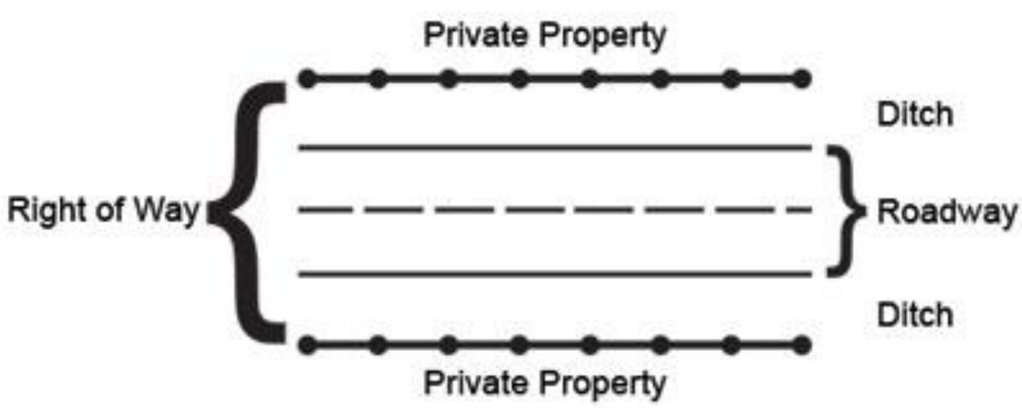


Figure 2: Schematic of typical roadway right-of-way section
Source: wilmorelandsurveying.com

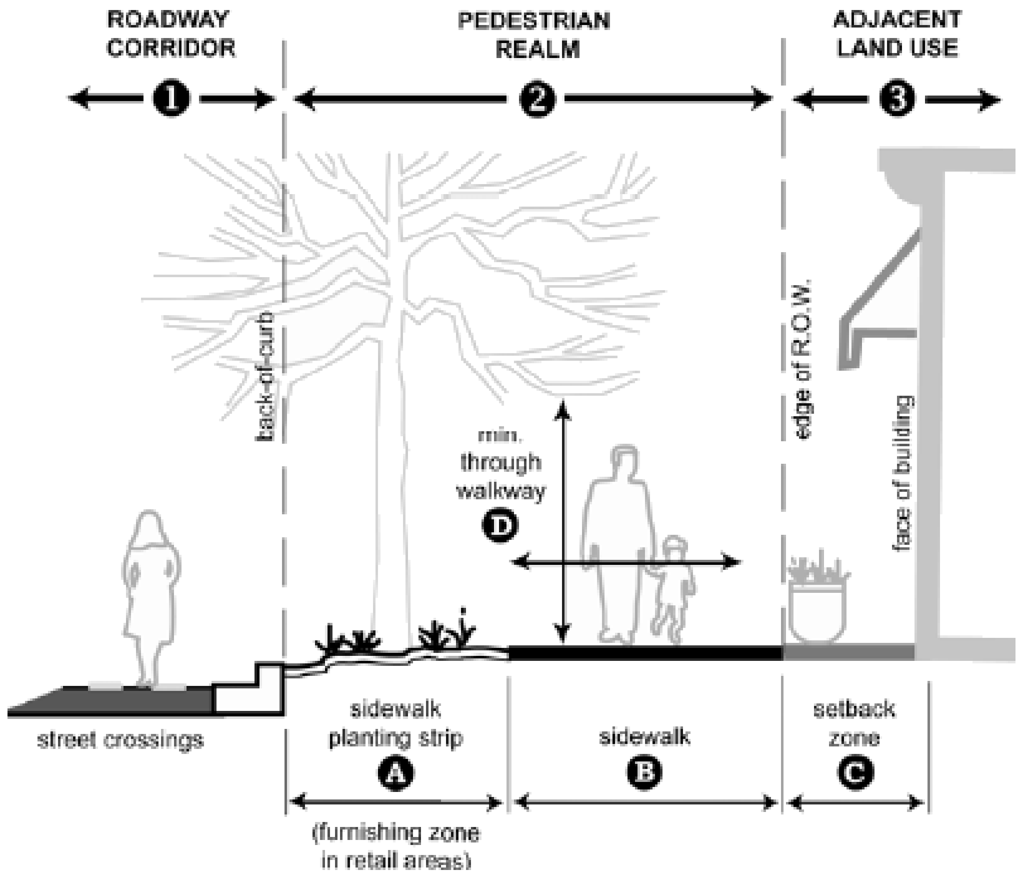


Figure 3: Schematics of roadway clearances and clear zones to typical roadside elements
Source: planetizen.com

The FHWA believes that the most responsible method for determining (bridge) roadway design standard is based on a consistent design approach, guided by past crash history and a cost-effectiveness analysis. The *Roadside Design Guide* provides guidance to help local agencies develop consistent design approaches for determining the widths of clear zones along roadways based on speed, traffic volume, roadside slope and curvature (AASHTO RDG 2011). The design guide also recommends clear zone ranges based on a width of 30 to 32 feet for flat, level terrain adjacent to a straight section of a 60 mph highway with an average daily traffic of 6,000 vehicles. For steeper slopes on a 70 mph roadway the clear zone range increases to 38 to 46 feet, and on a low speed, low volume roadway the clear zone range drops to 7 to 10 feet. For horizontal curves the clear zone can be increased by up to 50 percent. Another AASHTO publication, *A Policy on Geometric Design of Highways and Streets* (The Green Book), recommends a 10-foot minimum clear zone on collectors without curbs, low-speed rural collectors, and rural local roads (AASHTO Green Book 2004). For local roads and streets, a minimum clear zone of 7 to 10 feet is considered desirable on sections without curb. As a practical matter, the clear zone dimensions may be limited by available right-of-way; the location, frequency, and nature of roadside objects; the presence of valued resources such as wetlands; or the need to provide for pedestrians (AASHTO Green Book 2004). Thus, railing or guardrail countermeasures designed to full AASHTO standards may not be necessary/desirable for certain roadway characteristics such as LVRs. Nonetheless, this is still a concern on all roads.

Roadside crash fatality rate for rural roads is estimated to be nearly two and half times the average roadside fatal crash rate for all roads in the United States (NHTSA 2013), and these types of roads typically have very restricted rights-of-way, little to no clear zones, and substandard design features. Because of their low traffic volumes, drivers are more likely to become distracted and fatigued. Subsequently, the AASHTO *Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400)* (AASHTO GGDLVR 2001) is a tool used for addressing such roadside design issues on low volume roads. The guidelines state that traffic barriers are not generally cost-effective on roads with very low traffic volumes because the probability of striking a fixed object on these types of roads is extremely low when compared to similar higher volume roadways. However, the guidelines apply only to roads that are functionally classified as a local road and have a design ADT of 400 or less because of the high level of driver familiarity that is associated with these types of roads. The GGDLVR guidelines recommend that safety improvements should be initiated only when a safety problem exists at a site. Furthermore, the design guide states that a one-lane bridge can be used for roads with traffic volumes of less than 100 vehicles per day (vpd).

The *Manual on Uniform Traffic Control Devices* (MUTCD) is another set of guidelines which require all post/sign supports within the clear zone be made breakaway or shielded by a barrier (FHWA MUTCD 2009). Precisely, all supports located on highways posted at 50 mph or greater shall meet this criterion by January 2013. Likewise, the manual included that all existing supports located on roads posted at speeds 45 mph or lower, may meet this criteria when upgrading sign retroreflectivity or by 2019, whichever comes first (FHWA MUTCD 2009).

Engineering is not a science; it is an art. As an art, its practice precedes its theory. Thus, from the perspective of traffic safety and operations, most design countermeasures are routinely installed based on a subjective analysis of its benefits to the motorist. However, on occasion, it may not be immediately obvious of benefits gained from a specific safety design or treatment; thus, engineering judgement is also required to decide when, where, and how funds are spent to achieve maximum benefit. Addressing safety on local and rural roads presents several challenges including the actuality that: 1) safety issues are often random on local and rural roads; and 2) strategies to address local and rural road safety are diverse and draws from several safety areas (AASHTO RDG 2011). Consequently, per AASHTO's *Load and Resistance Factor Design* (LRFD) *manual*, agencies are required to develop its warrants for bridge installations (or upgrades) per site (FHWA LRFD 2001). Overall, agencies are also encouraged to upgrade existing safety hardware that has not been accepted either during its reconstruction or resurfacing, rehabilitation, or restoration (3R) projects or when the system is damaged beyond repair. The Roadside Safety Analysis Program (RSAP) offers an example of one methodological approach typically used for accomplishing a benefit/cost analysis of various countermeasure alternatives.

Another alternative for achieving maximum benefit is to consider countermeasures implementation using Context Sensitive Solutions (CSS). CSS encourages flexibility in the application of design standards and guidelines for projects to accommodate local concerns about issues such as community needs, environment and aesthetics. The Federal Lands Highway (FLH) Divisions, for example, have been practicing CSS for many years (FLH PDDM 2014). Sections 4 and 9 of its *Project Development and Design Manual* (PDDM) discusses the approach and application of CSS on Federal Lands' projects. The manual also recognizes that the full clear zones and barrier warrants recommended in the AASHTO *Roadside Design Guide* may not be practical to achieve particularly on rural low volume, low speed roads; thus, it offers guidance and guidelines to identify and treat serious roadside hazards accommodating this function of roadway. In light of CSS, the best decision will not always be to implement a recommendation from the manual. Although it is legitimate to exercise flexibility in the application of design standards and guidelines, it is also important to have a clear understanding of the safety consequences of context sensitive decisions so that an appropriate advantage can be achieved (FLH PDDM 2014).

Within the concept of (vehicle, roadway, and driver) safety, the FHWA have also developed strategic plans that are data-driven with a focus toward zero deaths and serious injuries involving roadway departure events. These policies are being achieved through various mediums, including the Strategic Highway Safety Plan (SHSP), the Highway Safety Improvement Program (HSIP), and the High Risk Rural Roads Program (HRRRP) to name a few. In collaboration with various stakeholders (including inter-state agencies and otherwise), the SHSP is a plan dedicated to addressing issues within the multidisciplinary 4E's of highway safety comprising of engineering (design and construction), education (licensing and awareness), enforcement (regulation and penalization), and Emergency Medical Services (response and incident clearance times).

Developed in 1981, the HSIP remains a federal-aid program guided to systematically identify, prioritize, and evaluate highway safety improvement needs. The HRRRP is similar in a sense that its aim is to systematically identify, prioritize, and evaluate safety improvement needs but on roadways functionally classified as rural major or minor collectors or rural local roads. Often, local road agencies do not have the resources needed to adequately address safety problems on the roads they own and operate; consequently they rely on federally funded plans and stakeholders to aid and guidance their efforts.

There are many (design) concepts that can assist designers in building the most suitable and sustainable bridge solutions including teamwork, rating systems, and innovation. Nonetheless, transportation officials and decision-makers also consider a life-cycle cost analysis as another important aspect/technique when assessing roadway/roadside investment projects decisions. From the perspective of bridge sustainability, several recent legislative and regulatory requirements recognize the potential benefits of bridge life-cycle cost assessment (BLCCA) viability and call for consideration of such analyses for infrastructure investments, including investments in highway bridge programs. According to the NCHRP Report 483: *Bridge Life-Cycle Cost Analysis*, BLCCA has received increased attention as a tool to assist transportation agencies in making investment decisions and in managing assets. Though the basic principles of life-cycle cost analysis were articulated more than 100 years ago, its systematic approaches did not appeared until 25 to 30 years ago in the United States (Hawk 2003). Hence, the report recognizes and establishes guidelines and standardizes procedures for conducting such life-cycle assessments.

Bridges last much longer than pavements. Thus, for a highway agency, bridges are a long-term investment. Throughout its useful life, a bridge requires both routine and periodic maintenance and occasional rehabilitation and replacement work. Bridges require a series of expenditures for various activities during their life cycles. The life cycle of a bridge plays an important role in determining the sustainability of the system. As illustrated in Figure 4, the report Guidance Manual also provides recommended guidance as to either the repair of existing bridges or for the evaluation of new bridge (cost-effective) alternatives per cycle period.

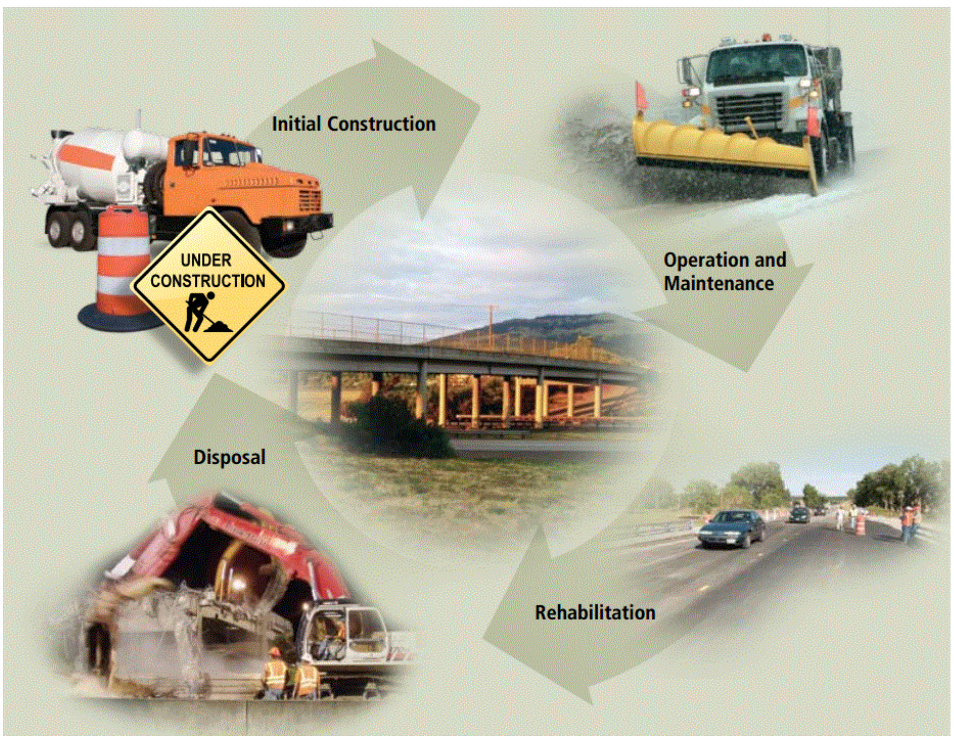


Figure 4: Typical life cycle of bridges
Source: aspirebridge.com

A life-cycle activity profile of a bridge can be represented by a series of future bridge activities laid out in a cash-flow diagram. It is typically expressed as equivalent present value (PV) of costs or as equivalent uniform annual costs, using compound interest and investments. Figure 5 illustrates an example of a typical BLCCA cash-flow diagram. It is necessary to make economic decisions with these future expenses in mind. Other costs generally accounted for in a bridge life-cycle cost (LCC) model can be expressed as the following (Hawk 2003):

$$\mathrm{LCC} = \mathrm{DC} + \mathrm{CC} + \mathrm{MC} + \mathrm{RC} + \mathrm{UC} + \mathrm{SV} \qquad \text{Equation (1)}$$

where:

LCC = life-cycle cost,
DC = design cost,
CC = construction cost,
MC = maintenance cost,
RC = rehabilitation cost,
UC = user cost, and
SV = salvage value.

In accordance with the manual, the calculation of life-cycle costs depends largely on the choice of a distinct time period over which operations and maintenance costs are accrued, discounted, and compared with capital costs. This time period is often referred to as the bridge's "design service life," and is typically decades long. Averaged over the time period of 10 to 30 years long, bridge design service life value can however sometimes be reduced substantially due to obsolescence or outdated features. A number of factors (technological, regulatory, economic, or social changes) could cause bridge obsolescence; nonetheless, they does not necessarily imply that the bridge is broken, inadequate, or otherwise dysfunctional (Hawk 2003). Rather, obsolescence simply implies that a bridge does not measure up to current needs, standard, or expectations. Other deteriorating factors of a bridge design life includes: bridge scouring and seismic vulnerability. Furthermore, according to Hawk, forecasting of bridge's obsolescence, scouring, and/or vulnerability are difficult at best and are certainly beyond the realm of current life-cycle cost-analysis models (Hawk 2003). However, effective life-cycle cost management should include explicit consideration of obsolescence.

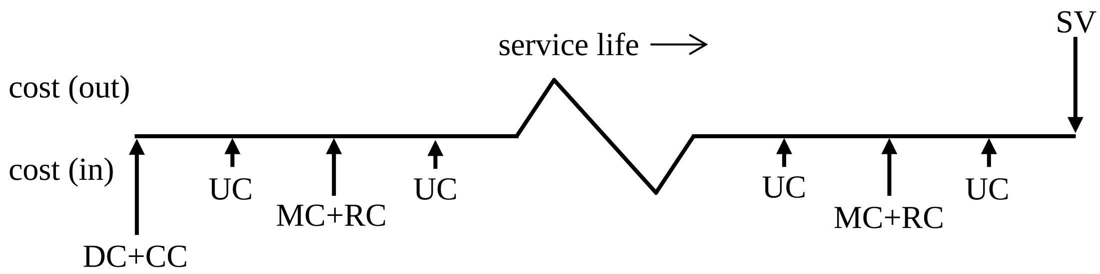


Figure 5: Example of a typical BLCCA cash-flow diagram
after (Hawk 2003)

Though expressed simplistically, the actual interpretations for some items in the model are different. In a bridge model, the user operating cost is often negligible, and the primary user costs (UC) are denial-of-use costs. Denial-of-use costs include the extra costs occurring during bridge congestion and repair, rehabilitation, and replacement. Major components of denial-of-use costs are value of time lost by the users because of delay, detours, and so forth.

2.2 Previous Studies

Supplemental to national policies and guidance, several agencies have analyzed design and safety countermeasures of bridges through various research efforts. Nonetheless, in design, most of the focus was typically placed on crashworthiness of the guardrail, bridge rail connections and end treatments. Hence, very few published literature sources were found to be directly related to the objectives of this research. Table 4 summaries several literary findings related to bridge and guardrail design characteristics.

Table 4: Summary of studies related to bridge and guardrail design

Author(s)	Emphasis	Scope	Major Finding(s)
Stephens Jr. (2005)	Barrier warrants, selection, and design	LVR	There are four (4) steps warranting process for roadside barriers selection and design.
Klaiber et al. (2000)	Bridge replacement alternatives	LVR	Steel-beam precast unit bridges show to be a viable low volume road bridge alternative.
Reid et al. (2011)	Blocked and non-blocked Guardrail	MGS	Standard (blocked) MGS performed better than non-blocked MGS.
Thiele et al. (2011)	Low-Cost, Energy-Absorbing Bridge Rail	MGS	Modified MGS configuration reduced bridge deck width and associated cost.
Rosenbaugh et al. (2015)	Mow-strip weak-post, W-beam guardrail	MGS Crashworthiness	Mow-strip MGS was crashworthy in asphalt, and could be installed in concrete to prevent pavement damages.
Abu-Odeh et al. (2015)	Short radius guardrail (SRG)	Roadside Safety Device Crashworthiness	Crash tested successfully to MASH and is very effective in capturing vehicles in short distances while using readily available components.
Marzougui et al. (2010)	Low-Cost, Energy-Absorbing Timber Guardrail	Roadside Safety Device Crashworthiness	A 75-ft x 20-ft recovery area is needed behind the terminal and SBT guardrail.

2.2.1 Bridge and Guardrail Design Studies

Chapter 5 of the AASHTO RDG contains roadside barrier layout and design guidance. To better apply its design criteria and processes, it is important to first understand the philosophy of the design concepts presented therein. As illustrated in Figure 6, there are nine (9) key variables (factors) that typically influence the design process of roadside barriers (Stephens Jr. 2005). Those include:

1) L_C: clear zone width, measured from the edge of the traveled way;
2) L_A: lateral distance from the edge of the traveled way to the back of the hazard;
3) L_3: lateral distance from the edge of the traveled way to the front edge of the hazard;
4) L_2: offset of the roadside barrier, measured from the edge of the traveled way to the front face of the barrier;
5) L_R: runout length; the stopping distance off the pavement, measured longitudinally from the upstream extent of the hazard along the edge of pavement;

6) L_S: shy line offset. Rigid objects such as roadside barriers close to the pavement tend to intimidate drivers, causing them to slow down or shift positions;

7) (a:b): barrier flare (optional); typically described as a ratio in the RDG;

8) L_1: tangent length of the barrier; defines the beginning point of the flare, measured from the upstream limit of the hazard;

9) X, Y: length of need (LON).

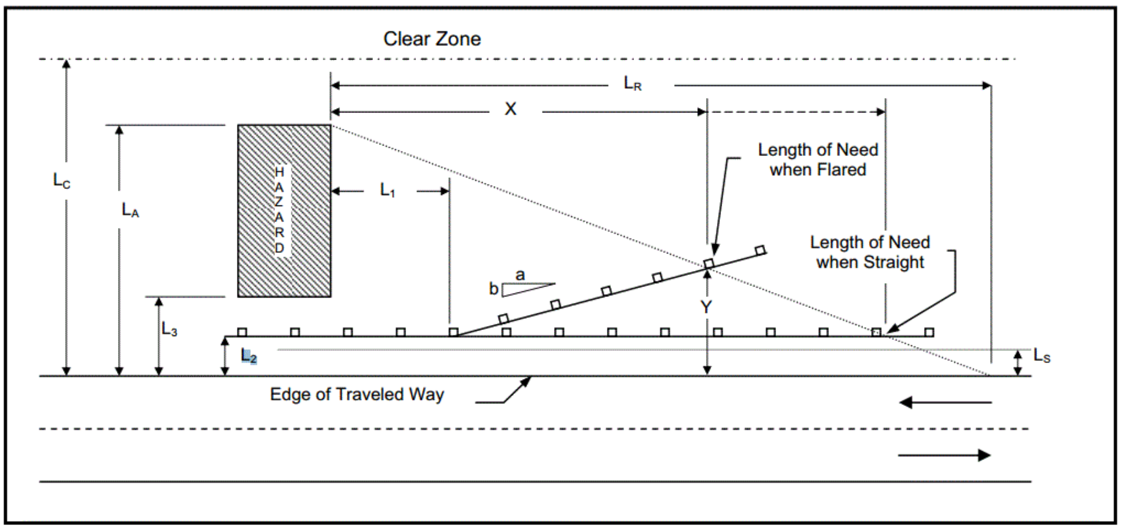


Figure 6: Roadside Barrier Design Variables
Source: (Stephens Jr. 2005)

Nonetheless, according to Stephens, warranting of roadside barriers is difficult to quantify, particularly for low-volume, low-speed roads. It is more a process to ensure that all important issues are addressed rather than a “cookbook” approach (Stephens Jr. 2005). Roadside barriers design process includes: 1) determining the needed clear zone, 2) identifying potential hazards, 3) analyzing strategies, and 4) evaluating barriers as effective countermeasures.

As exemplified in two studies conducted in Iowa, (Klaiber, Wipf and Nauman, et al. 2000) and (Klaiber, Wipf and Phares, et al. 2000), investigated two (design) alternatives for bridges on the Iowa secondary road system. The first concept (phase 1) considered steel beam precast units; whilst the second considered modification of a Beam-in-Slab Bridge concept. The steel-beam Precast Double Tee (PCDT) unit bridge presented in the first phase was used as the replacement structure at selected sites in Black Hawk County. Based upon the design, construction and service load testing, the authors found that the steel-beam precast unit bridge showed to be a viable low volume road bridge alternative (Klaiber, Wipf and Nauman, et al. 2000). Furthermore, they suggested that if salvaged beams are used, the initial bridge cost could be significantly reduced. Also, according to Klaiber, Wipf, Nauman, & Siow, the utilization of standard construction methods process could result in simple to use systems that can be completed with a typical bridge construction crew (Klaiber, Wipf and Nauman, et al. 2000).

The second concept (phase 2) constructed and tested full-scale composite beam, two-beam, and fatigue push-out samples. On the basis of the work completed, the research team recommended that the use of alternate shear connectors (ASC) in the Steel Beam Precast Units would further simplify fabrication (Klaiber, Wipf and Phares, et al. 2000). The authors concluded that prior to using this system in a demonstration bridge, the effects of increasing the distance between beams (which would reduce the number of steel beams required in a given bridge) and lowering the holes associated with the ASC in the beam webs (to improve the resistance to transverse tension forces caused by loading between the steel beams) will need to be further investigated (Klaiber, Wipf and Phares, et al. 2000).

Studies show that the Midwest Guardrail System (MGS) has been proven to provide exceptional redirective capability in standard and special applications according to the TL-3 safety performance evaluation criteria for both the NCHRP Report 350 and MASH (Reid, et al. 2011). More specifically, as a nonproprietary, strong-post, W-beam guardrail system consisting of standard steel or wood guardrail posts with blockouts that are 12 in. (305 mm) deep, roadway width acquirement for such a design approach is not always accessible. In response, a study led by Reid, et al., aimed to develop similar MSG without blockouts. The study was successfully crash tested per MASH criteria and compared results from 1100C small car and 2270P pickup truck tests for the standard (blockout) and nonblocked versions of the MGS. The authors concluded that the standard MGS performed better than the nonblocked MGS (Reid, et al. 2011). As a result, it is recommended that the nonblocked MGS be used only in places where roadway width is a limiting parameter. Nonetheless, if roadway width is not restricted, the use of blockouts as designated in the design drawings of the standard MGS is still recommended (Reid, et al. 2011).

Another study conducted by (Thiele, et al. 2011), developed and configured a new, low-cost bridge barrier system (Figure 7) compatible with the MGS. More specifically, the railing was designed to 1) be suitable for use on bridges or culverts with spans greater than 25 ft (7.62 m), 2) provide comparable lateral stiffness and strength to the standard MGS, 3) allow controlled post rotation when lateral loads become high, and 4) provide a yielding post or post-to-deck connection that mitigates bridge deck damage during most vehicular impacts. The barrier system was also configured to be simple, economic (reduced bridge deck width and its associated costs), and usable both on newly constructed bridges and for retrofitting to existing bridges.

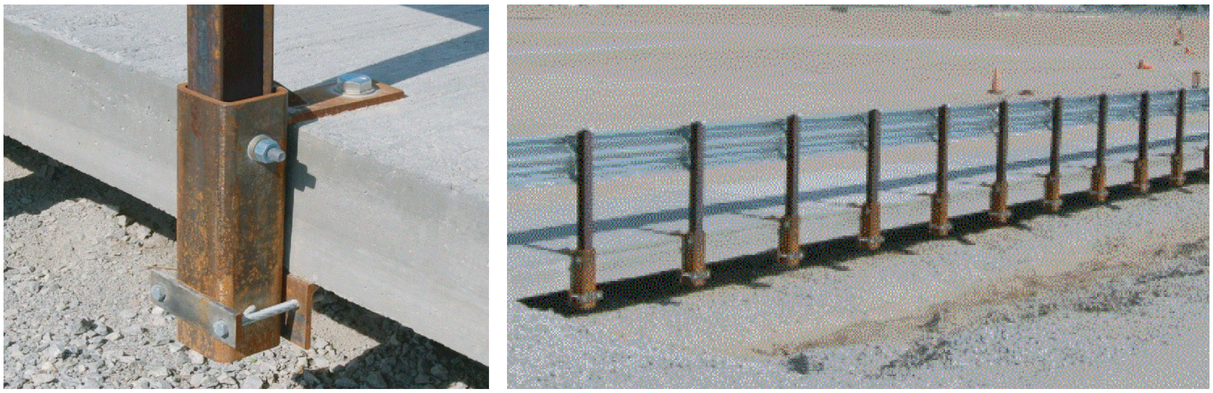


Figure 7: Modified MGS, overview (right) and close-up (left) of deck attachment assembly
Source: (Thiele, et al. 2011)

Several concepts including strong-post designs with plastic hinges and weak-post designs with bending near the bridge deck attachment for an energy-absorbing bridge post were also developed and tested. More details on the configurations and test results can be found in Figure 26 through Figure 29 of appendix A. In order to eliminate the need for approach transitions, the railing stiffness, strength, and deflection characteristics had to be comparable to those of guardrail posts embedded in soil. Furthermore, two full-scale crash tests were performed in accordance with the TL-3 impact conditions provided in the MASH. The bridge rail system met all safety performance criteria for both the small car and pickup truck crash tests. According to the authors, Barrier VII computer simulations, in combination with the full-scale crash testing programs for the bridge railing and MGS, demonstrated that a special-approach guardrail transition was unnecessary (Thiele, et al. 2011).

Similar study conducted by (Rosenbaugh, et al. 2015), evaluated the effectiveness of MGS weak-post, W-beam guardrail system for use within mow strips (asphalt and concrete) pavements. The evaluation began with three (3) rounds of dynamic bogie testing. According to Rosenbaugh et al, the first round showed that 4-in. (102-mm) thick concrete would sustain only minor spalling from impacts to the posts. However, the posts would push through 4-in. and 6-in. (102-mm and 152-mm) thick asphalt mow strips. During the second round, 24-in. (610-mm) long, 4-in. x 4-in. (102-mm x 102-mm) sockets with 10-in. x 9-in (254-mm x 229-mm) shear plates were utilized to better distribute the impact load to the asphalt pavement and prevent damage. Lastly, the third round, consisted of dual-post impacts, and the asphalt suffered from shear block fracture between the two 24-in. (610-mm) sockets and the back edge of the mow strip (Rosenbaugh, et al. 2015). Nonetheless, a dual-post test within a 4-in. (102-mm) thick concrete pad showed only minor spalling. According to the authors, full-scale MASH tests was conducted on the system installed within an asphalt. Due to the Round 3 results, the asphalt thickness was increased to 6 in. (152 mm), and the socket depth was increased to 30 in. (762 mm). Results showed that the 2270P pickup was contained and safely redirected, and all MASH safety criteria were satisfied (Rosenbaugh, et al. 2015). Unfortunately, the asphalt fractured, and a 2½-in. (64-mm) wide crack ran from socket to socket throughout the impact region of the system. Therefore, the authors concluded that the weak-post guardrail system was crashworthy, but would require repairs in its current configuration (Rosenbaugh, et al. 2015). Additionally, Rosenbaugh et al suggested that the system could also be installed in a concrete mow strip to prevent pavement damage.

In other instances when two roadways intersect surrounding restrictive features such as a bridge rail and/or waterway, it becomes difficult to fit a guardrail with the proper length, transitions, and end treatment along the traveled way. Possible solutions for such cases include relocating the constraint blocking the placement of the guardrail, shortening the designed guardrail length, or designing a curved guardrail; however, curved (or short radius) guardrails have shown to present the most viable solution, according to another study led by (Abu-Odeh, et al. 2015) at the Texas A&M Transportation Institute (TTI). To this end, the authors aimed to investigate, model, and simulate optimal short radius guardrails systems protocol to meet AASHTO's MASH recommended test levels parameters. Table 5 lists an example of some recommended test parameters for MASH TL-3, and Figure 8 shows their impact locations considered.

SRG systems used for these tests comprised of 12.5-ft curved W-beam section for the intersection system with 8-ft radius connected to 25-ft minimum straight W-beam section for the secondary road. It was then terminated with a positive anchor, allowing the beam to rotate. The primary road was connected to the curved section with a spliced short W-beam segment measuring 6.25 ft. Along this spliced section, two posts measuring 7⅞ × 7⅞ × 72 inches were embedded 44 inches into the soil. The modified design systems also consisted of 6-inch × 8-inch control release terminal breakaway posts, 10-gauge W-beams (or Thrie beams to improve trucks/cars stability), a rotating deadman anchor, a rail height of 31 inches, ASTM A307 button head bolts, and Texas DOT metal beam guard fence transition (Abu-Odeh, et al. 2015).

Table 5: Example of MASH TL-3 Recommended Test Matrix Parameters

Test Number	Vehicle Designation	Impact Speed	Impact Angle	Impact Tolerance (KE)
3-30	1100C	62 mph	0°	≥288 kip-ft
3-31	2270P	62 mph	0°	≥594 kip-ft
3-32	1100C	62 mph	5−15°	≥288 kip-ft
3-33	2270P	62 mph	5−15°	≥594 kip-ft
3-35	2270P	62 mph	25°	≥106 kip-ft

Source: (Abu-Odeh, et al. 2015)

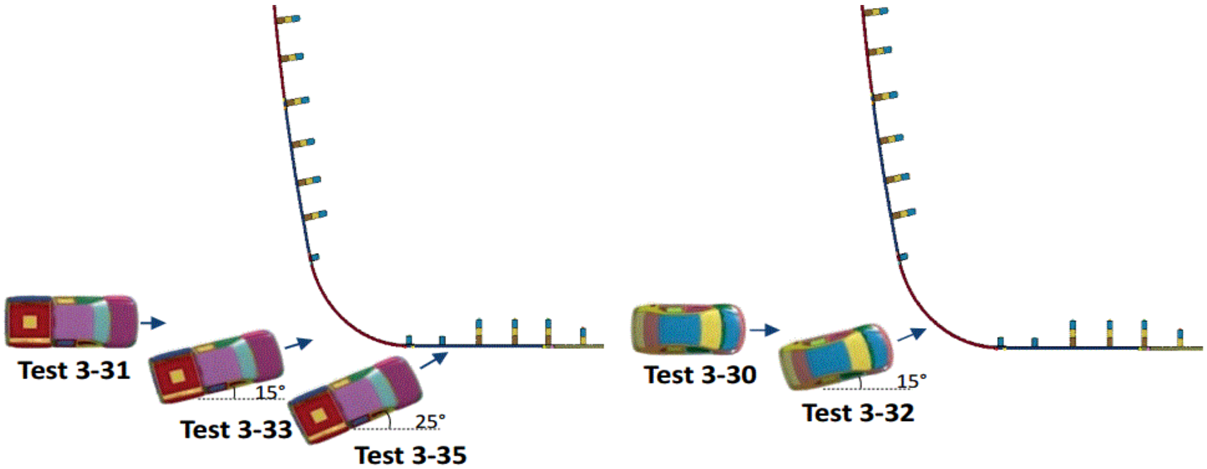


Figure 8: Example of MASH TL-3 Recommended Test Matrix Parameters
Source: (Abu-Odeh, et al. 2015)

In all, the study concluded that the use a single Thrie beam instead of two W-beams helped improve the interaction of the vehicle with the SRG system (Abu-Odeh, et al. 2015). More specifically, the system without sand barrels had vehicular instabilities. Also, the addition of sand barrels to the rail significantly helped to attenuate the energy of the impact (Abu-Odeh, et al. 2015). Nonetheless, the authors' recommended further detailed evaluations of the design concepts per site application.

Considering other systems such as timber bridges, a study led by (Marzougui, et al. 2010) investigated the crashworthiness of energy-absorbing gating design for an end-treatment to the Steel Backed Timber (SBT) Guardrail using computer simulations. As shown in Figure 9, in contrast to a traditional (buried in back-slope) SBT design, the energy-absorbing gating, when impacted, allows the end terminal to absorb the energy of the vehicle and bring it to a controlled stop before reaching the barrier start point. The gating effect also allows the end terminal to breakaway upon contact and redirect the errant vehicle when impacted at the length of need point (Marzougui, et al. 2010). The study documented a series of five full-scale crash tests of vehicles and various end-terminal designs to assess the barrier performance under different test conditions. Results from the models showed that the end-terminal design meets all NCHRP Report 350 in addition to MASH TL-2 safety performance recommendations (Marzougui, et al. 2010). Nonetheless, the authors noted that for this design to be more effective, a 75-ft x 20-ft recovery area is needed behind the terminal and SBT guardrail. Figure 10 shows example of a traditional buried in back-slope SBT guardrail.

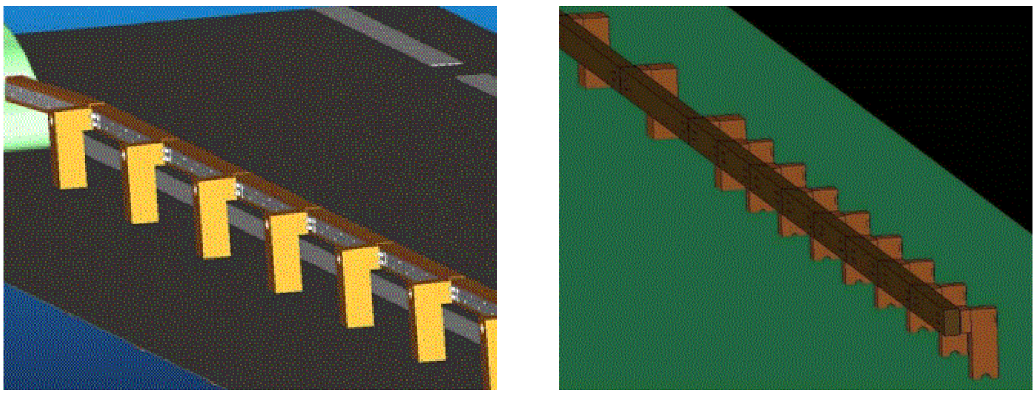


Figure 9: Models of Buried in Backslope (left) and Energy-Absorbing Gated (right) end-terminals
Source: (Marzougui, et al. 2010)

Figure 10: Example of a traditional SBT guardrail with buried in backslope end-treatment
Source: wsdot.wa.gov

2.2.2 Bridge and Guardrail Safety Operations Studies

From the perspective of traffic safety and operations, though there are standards and policies established to aid and abet roadside designs and overall safety, experience and research improvements overtime has also helped mitigate roadside safety problems.

Once in place, there are many fixed objects that present some degree of risk if struck but are not serious enough to consider removal or shielding countermeasures. It is important to first understand the philosophy of roadside design concepts to better apply its criteria and processes. Due to the complex nature of roadside barrier warranting, particularly for low-volume, low-speed roads, Stephens suggests special, practical considerations be taken for such road classification per condition situation (Stephens Jr. 2005). Relatively to this research, those includes consideration for speed, hazard offset and special design considerations for aesthetics and severe conditions. Table 6 lists hazards and their potential severity. Severity increases from 1 to 3, with Group 3 being the most severe. Furthermore, Stephens noted that considerations should also take into account both the cost of a barrier and the expected crashes into that barrier. Often, local conditions, policies, and resources are also considered. In all, these considerations lead to a list of technically acceptable barriers for a specific site.

Table 6: Fixed Objects Potential Hazards

Potential Hazard	Group 1 (Low Severity)	Group 2 (Moderate Severity)	Group 3 (High Severity)
Bridge piers, abutments and railing ends			X
Boulders, less than 0.3 m (1-ft) in diameter		X	
Boulders, 0.3 m (1-ft) in diameter or larger			X
Non-breakaway sign and luminaire supports		X	
Individual trees, greater than 100 mm (4 in) and less than 200 mm (8 in) diameter	X		
Individual trees, greater than 200 mm (8 in) diameter		X	
Groups of trees, individually greater than 100 mm (4 in) diameter[1*]			X
Utility poles		X	

Source: (Stephens Jr. 2005)

Figure 11: Unshielded Bridge Rail End
Source: (Stephens Jr. 2005)

Specifically, within the perspective of safety operations, the concepts of probability and severity must be understood to effectively evaluate roadside safety alternatives. However, it is impossible for regression models to account for each and every factor associated with crash occurrences (Persaud and Dzbik 1993). Nonetheless, a study by (Stephens Jr. 2005) suggests that

[1*] Because of driver expectancy, a group of trees at a consistent offset for lengthy distances may experience lower encroachment rates, even though the offset may be within the clear zone. In such instances, it may be appropriate to consider the trees a Group 2 hazard.

the probability (or likely frequency) of a vehicle striking any roadside object or condition (including barriers) should be determined by a complex set of variables, including:

- Traffic volume,
- Speed,
- Roadway characteristics (number and width of lanes, shoulders, divided or not, etc.),
- Horizontal curvature,
- Grade,
- Size and offset of the hazard or barrier, and
- Rate of encroachment (affected by familiarity of drivers, driver distractions, driver expectancy and design consistency of the roadway).

Table 7 summaries several literary findings related to bridge and guardrail safety operations characteristics.

Table 7: Summary of studies related to bridge and guardrail safety operations

Author(s)	State	Scope	Major Finding(s)
Stephens Jr. (2005)	(USA)	Barrier warrants, selection, and design	Traffic volume, speed, roadway characteristics, including grade and curvature all affect the odds of a crash.
Mehta et al. (2015)	Alabama	Bridge Components Safety Performance	High presence of trucks and use of transition railings were found to be significant noncontributory factors associated with bridge crashes.
Zou et al. (2014)	Indiana	Traffic Barriers Safety Performance	Guardrails should be preferred over concrete barriers, and cable barriers should be preferred over guardrails where geometric conditions allow.
Bigelow et al. (2011)	Iowa	Guardrail and Bridge rail Performance	Frequency of vehicular crashes are more prevalent on bridges with smaller widths in relation to its roadway width.
Seitz and Salfrank (2014)	Kansas	Guardrail and Bridge rail design	Railings installed on new bridges could be of a non-tested design if the structure meets the set of conditions.
Fitzpatrick et al. (2014)	Massachusetts	Driver Behavior and roadside clear zone size	Drivers drive closer to the edge of road and increase speeds as the size of the clear zone increase.

Gates and Noyce (2005)	Minnesota	Guardrail Barrier Effectiveness	Guardrails installed at all four quadrants of a bridge has a B/C ratio ranging from 3.99 to 6.62 and is cost-effective at ADT greater than 400 vpd.
Dare (1992)	Missouri	Guardrail Barrier Effectiveness	Roads with ADT of 400 vpd, at 60 mph, and 2-ft lateral offset do not have sufficient traffic volumes to warrant approach guardrail.
Hall (1982)	New Mexico	Guardrail Barrier Effectiveness	Guardrails used to protect abutments or bridges would reduce the crash severity by 50 percent.
Turner (1984)	Texas	Bridge Components Safety Performance	Bridges became “safer” as one moves from negative to positive relative widths of bridges.
Holdridge et al. (2005)	Washington	Performance of Roadside Object Safety	Effect of crash barriers on fatalities is rather minor due to the rareness of fatal crashes with these barrier types.
Lee and Mannering (1999)	Washington	Safety Performance of Roadside Objects	Perhaps, roadside recovery space is the most important factor in reducing crash severity in mix to narrower widths.
Delaney et al. (2002)	(Australia)	Fixed Objects vs. Non-fixed Objects	Trees and utility poles were the roadside objects struck more frequently and produced more severe injuries.
Ray and Hiranmayee (2000)	--	Impact Risk Involving Roadside Objects	The maximum difference between impact velocities should be no greater than 10 m/s for head injury.

One study (Mehta, et al. 2015) conducted in 2014 aimed to develop safety performance functions for overall crashes and single-vehicle crashes (SVC) involving bridges in Alabama. The study focused on 1,122 bridges located on state and interstate highways, including ramps. Of the bridges considered, 9,985 crashes along the bridge overpass were associated and used as bridge-related crash incidents for the analysis. The study used negative binomial regression to estimate crash frequency involving bridges in addition to identifying factors associated with bridge crashes. Best fitting models were chosen using log-likelihood and AIC (Akaike Information Criterion) values. Of all variables considered, annual average daily traffic (AADT), bridge length, shoulder width, and the use of approach railings/guardrail-ends were identified to be significant contributing factors associated with bridge crashes. Whereas, high presence of trucks and use of transition railings were found to be significant noncontributory factors. These factors showed evidence to decrease the expected number of bridge crashes. It was also found that the predictive capability of the final model (using all significant variables) was not much different from the predictive capability of similar model using only AADT, bridge length, and truck percentage. Moreover, the authors noted that if available, the variables related to the presence of bridge railings or guardrails may be included, but are not essential (Mehta, et al. 2015).

Another (Zou, et al. 2014), analyzed injuries sustained by vehicle occupants when colliding with several types of roadside barriers along freeways. The study focused on the safety performance of road barriers in Indiana in reducing the risk of injury. In so doing, the authors compared the risk of injury among different hazardous events faced by an occupant in a single-vehicle crash. The studied hazardous events included rolling over, striking three types of barriers (guardrails, concrete barrier walls, and cable barriers) with different barrier offsets to the edge of the travelled way, and striking various roadside objects. A total of 2124 single-vehicle crashes (3257 occupants) that occurred between 2008 and 2012 on 517 pair-matched homogeneous barrier and non-barrier segments were analyzed. Conclusive results drawn from the study indicated that crashes involving barriers such as guardrails or cable barriers are typically less severe than crashes with poles or rollover crashes. More specifically, the study found that the likelihood of occupant injury reduced significantly across several crash barrier types as offset distance increased depending on the barrier struck (Zou, et al. 2014). For example, odds of injury decrease 43% when colliding with a guard rail within 15-18 feet rather than colliding with a median concrete barrier within the same lateral offset distance. Further injury reductions were observed when compared to a concrete barrier within 7-14 feet of the traveled way. The study claimed that guardrails should be preferred over concrete barriers, and cable median barriers should be preferred over guard rails where geometric conditions allow (Zou, et al. 2014). The study however noted that there was a certain degree of invariability across vehicle characteristics in regards to crash severity sustained and general interactions between barrier types during collisions.

Most recent study (Bigelow, Hans and Phares 2010) in the state of Iowa concerning low-volume roads traffic barriers was aimed to determine criteria and guidelines used by states for bridge and approach guardrail implementation on low volume roads. The primary objective of the study was to provide information about the use of bridge rail and approach guardrail on LVR in Iowa. Specifically, the use of descriptive, statistical, and economic analyses were used to aid the investigation. The authors found that, based on a survey of non-Iowa bridge owners, most agencies tend to not use ADT as a requirement for bridge barriers; however, majority did use protective treatments other than W-beam as effective countermeasures (Bigelow, Hans and Phares 2010). Regardless, the criteria for determining traffic barrier use for most agencies however have not changed over the past 10 years. Within the Iowa structure and crash databases, the analyses provided suggested that crash rate decreased as bridge traffic volume (or bridge width) increased – both the crash frequency and crash rate were higher for bridges with lower traffic volumes (Bigelow, Hans and Phares 2010).

Currently, the Iowa Department of Transportation (Iowa DOT) provides Instructional Memorandums (IowaDOT IM 2013) to Local Public Agencies which recommends bridge railing upgrades protocol for determining the need for traffic barriers. A section of this document (IM 3.213) is provided in appendix A. Additionally, this document warrants upgrade standards for roadway bridges and culverts based on scoring of five criteria. Those including:

1. Crash History (in the past 5 years),
2. ADT (current year annual daily traffic),
3. Bridge Width (curb-to-curb) in feet,
4. Bridge Length (in feet), and
5. Bridge Type.

A previous study led by Schwall (Schwall 1989), in Iowa in 1988, looked at the cost-effectiveness of approach guardrails on primary-system roads. Schwall found that, to obtain a B/C ratio of 1.0, or better, a traffic volume of at least 1,400 vpd with a guardrail offset of 2-ft is required.

Later study was conducted by Seitz and Salfrank, in 2014, to better maximize the safety benefits of low-cost bridge design for low-volume local roads in Kansas given its limited funding (Seitz and Salfrank 2014). The study consisted primarily of bridge/approach guardrail crash-cost analyses. In conclusion, the authors recommended that bridge rails installed on new or rehabilitated bridges utilizing federal funds could be of a non-tested design if the structure meets the set of conditions (Seitz and Salfrank 2014). Furthermore, Seitz and Salfrank suggested that this non-tested design should be constructed of a w-beam guardrail section mounted on standard guardrail posts that are fastened to the bridge structure either by welding or a bolted connection. In addition, no approach guardrail will be required on these bridges. Nonetheless, they noted that, although the findings would support a policy that does not require installation of bridge rails on bridges between 20 ft. and 50 ft. on roads functionally classified as Local Roads with less than 50 vehicles per day (vpd), it is recognized that there are benefits of the rail that cannot be evaluated by this effort (Seitz and Salfrank 2014).

In support, earlier studies in Kansas, (Russell and Rys 1998), compared the probabilities and expected cost of crashes at bridge and culvert locations with bridge rails and headwalls vs. the expected cost of crashes with bridge rails and culvert headwalls removed. Russell and Rys concluded that the expected costs of these crashes were less with the concrete rails and headwalls removed for ditch depths of 2.4 meters or less (Russell and Rys 1998).

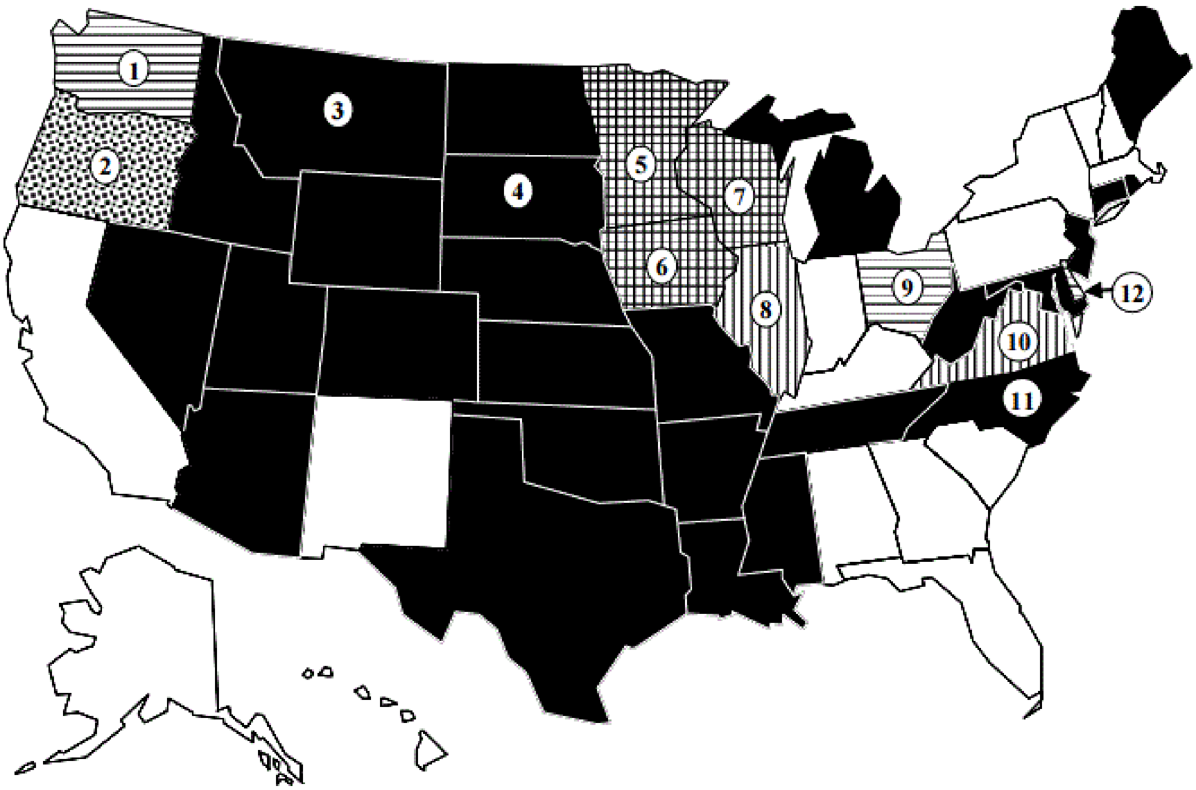
Other research (Fitzpatrick, et al. 2014) have also shown that natural landscapes can effectively lower crash rates and cause less frustration and stress to the driver. According to a study by Fitzpatrick et al., the presence of roadside vegetation can serve as an effective countermeasure which also provides both environmental and psychological benefits to road users and drivers in particular. The study explored relationships between the sizes of clear zones and the presence of roadside vegetation with respect to vehicle speed and vehicle lateral positioning. An evaluation of 100 licensed drivers on speed selections for both real and virtual roads containing four combinations of clear zone sizes and roadside vegetation densities was utilized and validated using field data collection. Vehicle speeds and lateral positions were the primary metrics used to evaluate drivers' response. In result, the authors observed that drivers' virtual road speeds for roadways with medium clear zone/dense vegetation or large clear zone/spare vegetation correlated with real road speed field data (Fitzpatrick, et al. 2014). The study confirmed that drivers speed selections increased as the size of clear zone increased. Additionally, they concluded that drivers tend to drive closer to the edge of road as the size of the clear zone increase (Fitzpatrick, et al. 2014). In all, the study successfully demonstrates the relationship between clear zone design and driver behavior, which could improve clear zone design practices and thus roadway safety.

An earlier study (Gates and Noyce 2005) analyzed characteristics of 96 run-off-road rural-area crashes that occurred on the approach or departure railings of low-volume state-aid highway bridges in Minnesota over a 14-year period from 1988 to 2002. Gates and Noyce noted that because many bridges and roadways on local systems are built and/or maintained using state funds, they typically are required to conform to state standards and guidelines (Gates and Noyce 2005). Figure 12 shows a survey response of the state-of-the-practice for bridge approach guardrail installation on low volume highways for various agencies, including those maintained by counties and other local jurisdictions. The objective of the study was to determine the ADT at which the benefit-cost ratio suggests that installation of guardrails at the bridge approach is cost-effective (i.e., B/C > 1.0). On the basis of statistical and benefit-cost analyses, the study confirmed that crashes that occurred at bridges with approach guardrails were significantly less severe than crashes that occurred at bridges without guardrails (Gates and Noyce 2005). Crashes involving bridges with approach guardrails were more likely to result in property damage only. More precisely, approach guardrails installed at all four quadrants of a bridge had a benefit-cost (B/C) ratio ranging from 3.99 to 6.62 and are cost-effective at traffic volumes greater than or equal to 400 vehicles per day (vpd) (Gates and Noyce 2005). The study recommended to the Minnesota Department of Transportation (MnDOT) that a minimum threshold of 400 vpd be a requirement for the installation of a bridge guardrail on low-volume roads (LVR) which is consistent with current roadside clear zone guidelines suggested by AASHTO for local LVRs. The authors furthermore suggest that bridges with ADT volumes between 150 to 400 vpd be reviewed individually because bridges with unique circumstances (bridges along curves and/or bridges with narrow widths) may warrant guardrails. Guardrails along bridges serving ADT less than 150 vpd is considered probably not cost-effective according to Gates and Noyce;

nonetheless, if a guardrail is installed, it should be on all four corners of the bridge (Gates and Noyce 2005).

The Missouri Highway and Transportation Department (MoDOT) also concluded from a study by Dare in 1992 that roads with an ADT of 400 vpd, at 60 mph speed limit, and 2-ft lateral guardrail offset do not have sufficient traffic volumes to warrant approach guardrail (Dare 1992). The same study also provided higher thresholds values for 40 and 50 mph speeds and lateral offsets of 8- and 10-ft respectively.

Approach guardrail is commonly used countermeasure designed to prevent collisions of errant run-off-road vehicles with roadside fixed objects such as bridge rails. In a late 1970's study for the New Mexico DOT, (Hall 1982) found that guardrail accidents were more often characterized by rural conditions, unfamiliar drivers, and snow-covered roads. Specifically, according to Hall, collisions with guardrail produced severity indices that were approximately 50 percent lower than that of collisions with bridge abutments, which had the highest severity index of all fixed object collisions that were examined (Hall 1982). As a result, it was suggested that the addition of guardrail to protect an abutment or bridge would reduce the crash severity by 50 percent. In retrospect, however, Hall also found that bridges were the most common location for a guardrail crash to occur due likely to the fact that bridges were the most common location for guardrail installation with 31 percent of all installations (Hall 1982).



Legend:

- All state-aid bridges protected (n = 26)
- ADT and speed threshold (n = 3)
- ADT threshold (n = 2)
- Decision made on case-by-case basis (n = 1)
- Speed threshold (n = 3)
- No response (n = 15)

Notes:

1. Historic bridges or urban routes 35 mph or less may be exempted
2. ADT and operating speed are used to make bridge approach guardrail decisions for low-volume state and local roads.
3. Very low volume roads or low speed urban facilities may be exempted on a case-by-case basis, but rarely is this done.
4. ADT < 150 requires only turned down guardrail treatment connected directly to bridge
5. ADT < 750 doesn't require guardrail; design speed 40 mph or less may also be exempted
6. Guardrail not needed if speed limit is 35 mph or less; also exempted are bridges with ADT < 200, bridge wider than 24 ft, on tangent, and benefit/cost ratio < 0.8
7. ADT < 300 doesn't require guardrail; also exempted are curbed urban roads with design speeds 45 mph or less
8. ADT < 150 doesn't require guardrail (tangent alignment only); approach guardrail required for all bridges on curves
9. Speed limit 45 mph or less doesn't require guardrail
10. ADT < 400 doesn't require guardrail; factors such as speed, crashes, paving, cost, and bridge rail condition are also considered
11. Very low volume roads or low speed urban facilities may be exempted on case-by-case basis, but is generally not done
12. Design speeds 35 mph or less use a tapered-down parapet; guardrail must be applied if slope steeper than 3:1

Figure 12: Criteria for approach guardrail application on state-funded local roads

Source: (Gates and Noyce 2005)

Similar study conducted by Turner aimed to identify hazardous structures, evaluate potential safety treatments, predict bridge accidents, and set priorities for improvement at bridges in Texas (Turner 1984). Rural, two-lane two-way bridge crashes were the focus of the study. The investigation was narrowed to a statistically consistent sample of 2,849 crashes that occurred at or near 2,087 bridges during a 4-year period. The research led to emphasis on three key variables: (1) relative width of a bridge (bridge width minus road width), (2) ADT, and (3) width of the approach roadway. These variables were used to develop a probability table capable of prediction collision. Results showed that bridges became "safer" as one moves from negative to positive relative widths of bridges (Turner 1984).

According to another study by (Holdridge, Shankar and Ulfarsson 2005), crashes involving fixed-object are of concern to engineers and those in the transportation industry; specifically, bean-guardrail lead ends, bridge rail lead ends, trees, light poles and traffic poles all increase the likelihood of fatal injuries. Conversely, the authors noted that barrier face collisions (i.e. concrete barrier, and bean-guardrails) tend to degrade potential injury severities from incapacitating (or evident) injuries to non-incapacitating injuries. The study found the effect of crash barriers on fatalities to be rather insignificant due to the rareness of fatal crashes with these barrier types (Holdridge, Shankar and Ulfarsson 2005). Unexpectedly, the study found that inclement environment conditions tend to decrease run-off-road crash severity, largely due to the relative decrease in driving speeds exhibited by road users in poor conditions. Crashes occurring near underpasses were found to have higher propensities of resulting in non-injury severities (Holdridge, Shankar and Ulfarsson 2005). Lastly, the study concluded that, as speed limits increase, so does injury crashes.

In like manner, a study (Lee and Mannering 1999), in Washington State, investigated the relationships among roadway geometry, roadside characteristics, and run-off-roadway accidents and concluded that temporal, environmental, driver-related, roadway, and roadside geometric characteristics all play a role in roadside crash severity. Specially, the study also declared that perhaps, roadside recovery space is the most important factor in reducing crash severity (Lee and Mannering 1999). Nonetheless, other factors such as driver inattention, lack of experience, and impaired driving create higher risks of severe injury crashes. Lee and Mannering acknowledged that due to the cost associated with roadside data collection, it is difficult to develop effective models for the relationship between run-off-road crashes and crashes involving fixed-objects. Some notable findings included: decreased crash severity when narrow shoulders are present, increased probability of fatal crashes on or near bridges, increased crash severity in the presence of tree groups, and decreased probability of incapacitating (or fatal) crash severity when utility poles are present (Lee and Mannering 1999). It would seem unconventional that crashes in the presence of utility poles would lower probability of severe injury; nonetheless, Lee and Mannering suggested that this could be due to the increased distance (recovery space) from the outside edge of traveled lane to the utility pole.

From a slightly new perspective, one study (Delaney, et al. 2002), aimed at investigating the nature and extent of motor vehicle collisions involving roadside fixed objects. This study was conducted for both metropolitan Melbourne and regional Victoria, thus accounting for both urban and rural environment. Chi-square test was performed to compare crash severities of crashes involving fixed objects with those involving non-fixed objects. As anticipated by the authors, a large number of crashes resulting in fatalities or severe injuries involved fixed objects more than non-fixed objects (Delaney, et al. 2002). A number of other crash characteristics were also examined (i.e. the type of fixed objects struck, vehicle types involved, speed limit of the road segment, etc.). Results from the study confirmed that trees and utility poles were the roadside objects struck more frequently and produced more severe injuries (Delaney, et al. 2002). In closing, the authors also noted that speed limit of the roadway had a significant effect on crash frequency as well as crash severity.

Another conducted by (Ray and Hiranmayee 2000), assessed the types of injuries sustained by vehicle occupants during side impact with roadside objects. The study was justified on the basis that severity of damage depends on the initial position of occupant relative to vehicle, velocity of the occupant and the response of the inner door and the object getting struck. Three injury types, namely head injury, thoracic injury and pelvic injury, relating to velocity-time history of intrusion were analyzed. To assess head injury, a value for head injury criteria (HIC) was calculated and a value of 1000 was considered the threshold at which linear skull fracture would start showing up. Next, thoracic injury was expressed through thoracic trauma index (TTI) which measured the average peak acceleration of the thorax via an Anthropometric Test Device (ATD). Lastly, peak lateral acceleration of the pelvis must not be more than 130g for acceptable performance. To conclude, the study recommended that the maximum difference between impact velocity of the vehicle and the velocity of the impacted face of the object getting struck should not be greater than 10 m/s, 9 m/s, and 13 m/s at every point on the velocity-time history of the struck object 10 milliseconds after establishing first contact with the roadside object for head injury, thoracic injury, and pelvic injury criteria respectively (Ray and Hiranmayee 2000).

2.2.3 Bridge and Guardrail Sustainability Studies

In recent decades, the concept of bridge resilience and sustainability have become a very sensitive subject matter in all phases of bridge life cycle, particularly in design and operational maintenances. They comprise significant amount of assets for which (transportation) agencies are responsible for. Agencies using federal funds often must conduct LCCA to justify their planning and design decisions, because of federal agencies requirements (Hawk 2003).

A host of life-cycle cost models is available in literatures. More specifically for transportation-related applications, life-cycle cost models have been developed for equipment management (Hatami and Morcous 2013), steel-bridge corrosion-protection systems (Tam and Stiemer 1996), lane marking (Miller 1992), rural advanced traveler information systems (Ullah, O'Neill and Bishop 1994), and transit fleet management ((Sherman and Hide 1995) and (Karlaftis, et al. 1997)). Table 8 summaries several literary findings related to bridge and guardrail sustainability characteristics.

Table 8: Summary of studies related to bridge and guardrail sustainability

Author(s)	Emphasis	Scope	Major Finding(s)
Hawk (2003)	Legislations	BLCCA	Agencies using federal funds often must conduct LCCA to justify their planning and design decisions.
Bocchini et al. (2014)	Resilience and Sustainability	BAS	Bridge resiliency, sustainability, and their combination should be analyzed with a probabilistic perspective.
Hatami and Morcous (2013)	Equipment Management	BAS	Deck widening has a lower net present value than deck replacement with regard to bridge costs and service life.
Mohammadi et al. (1995)	Optimal Timing and Efficiency	BAS	Bridge condition rating, works costs, and bridge service life expectancy are three major elements for bridge assessment.
Jiang and Sinha (1989)	Optimal Timing and Efficiency	BAS	Life-cycle cost assessment can also be used to determine optimal timing and efficiency of bridge activities.
Miller (1992)	Lane Marking	BMS	Cost of lane marking with aggregate crash costs by roadway type can yield safety benefits up to $60 per dollar spent on pavement striping, on average.
Aldemir-Bektas (2015)	Bridge element Inspection (BEI)	BMS	Consistency among element inspections will provide quality input for bridge condition assessment, performance measures, and overall decision-making.

Ikpong and Bagchi (2015)	Bridge Resilience Indicators (BRI)	BMS	Beyond the usual bridge condition monitoring, bridge resiliency includes the basis of replacement cost, failure consequence, and user cost.
Washer et al. (2014)	Risk-Based Inspection (RBI)	BMS	RBI practices differ from traditional approaches because the setting of inspection frequencies (or intervals) are not fixed or uniform.

In support of succeeding legislations and regulatory requirements (i.e. the *Intermodal Surface Transportation Efficiency Act of 1991* (ISTEA), the *Federal Executive Order 12893* (Principles 1994), and *The National Highway System Designation Act of 1995*, to name a few), research studies including (Bocchini, et al. 2014) in particular, suggested the prominence of structural and infrastructural sustainability particularly became of issue as of the late 1980s, when the construction industry introduced sustainable building assessment systems with weighted environmental, economic, and social aspects for buildings over their life cycles (Bocchini, et al. 2014). The study concluded that though very similar to one another, bridge resiliency or sustainability are two paramount complementary attributes of infrastructures. Only the combination of both concepts provides a truly comprehensive assessment of the quality of infrastructure (bridge) systems (Bocchini, et al. 2014). The authors noted that because of their quantitative metrics, it’s often a challenge to measure or collect data required for such assessment and thus, (resiliency, sustainability, and their combination) should be analyzed with a probabilistic perspective.

Many states and local agencies employ life-cycle cost analysis in their bridge-management systems as a means of decision-making in maintenance, rehabilitation, and replacement. One recent study, (Hatami and Morcous 2013), uses deterioration modeling strategies to develop deterministic and probabilistic LCCA of bridge components management for bridges in Nebraska. The deterioration analysis was performed for various bridges at several conditions (from condition 9, excellent; to condition 4, poor). Based on their results, Hatami and Morcous concluded that silica fume and epoxy polymer overlays had the lowest net present value compared to polyester overlay on bare deck at condition 6 and condition 7 respectively (Hatami and Morcous 2013). Furthermore, Hatami and Morcous concluded that replacing abutment expansion joints at the same place has a lower net present value than relocating them at the grade beam despite the fact that it causes deterioration of girder ends and bearings at a higher rate. Lastly, they suggested that deck widening has a lower net present value than deck replacement with regard to bridge costs and service life (Hatami and Morcous 2013).

Other studies, (Mohammadi, Guralnick and Yan 1995), have estimated such metrics using single parameter to quantify the bridge decision-making process in an optimal scheduling scheme. Based on current methods for LCCA implementation, three major elements including (1) bridge condition rating, (2) costs associated with various bridge works, and (3) bridge service life expectancy, were combined to estimate the parameter for bridge life-cycle cost assessment (Mohammadi, Guralnick and Yan 1995). Equation (2) demonstrates as follows its parameters:

$$\mathrm{BVI} = \mathrm{F}(\mathrm{r}, \mathrm{c}, \mathrm{t}) \qquad \text{Equation (2)}$$

where:

BVI = bridge value index,
F = objective function,
r = bridge condition rating,
c = costs, and
t = bridge service life expectancy.

Based on the premise of discounted future cash-flows activities and bridge service conditions, equation (2) can be used as an objective function in a mathematical-optimization scheme to justify expenditure benefits during decision-making processes. The authors proposed that although equation (2) is suggested for project-level analysis, it can be extended for use in network-level analysis (Mohammadi, Guralnick and Yan 1995). Nonetheless, life-cycle cost assessment has also been used to determine optimal timing and efficiency of bridge activities via systematic optimization BAS modeling (Jiang and Sinha 1989).

Other literature, including (Miller 1992), considered lane marking applied with fast-drying paint or thermoplastic, which are the most frequently used marking materials in the United States, as effective countermeasures in regard to bridge management and safety. Striping with fast-drying paint, suggested to cost $0.035/linear-ft. in rural areas and $0.07/linear-ft. in urban areas, were the basis of Miller's analysis. He recommended that though thermoplastic lines cost more than painted ones, they however can have lower life-cycle costs; he also included that in areas where snowplowing is unnecessary, they have longer lives (Miller 1992). Furthermore, Miller acclaimed that given that existing longitudinal pavement markings reduce crashes by 21%, and edgelines on rural two-lane highways reduce crashes by 8%, applying these percentages to aggregate crash costs by roadway type can yield safety benefits up to $60 per dollar spent on pavement striping, on average (Miller 1992). Lastly, he noted that these benefits rise with traffic volume, however, are twice as beneficial in urban areas than rural areas (Miller 1992). Nonetheless, in the assessment of the alternatives for a bridge countermeasures, crash costs of users are often significant because bridges correlate with crash rates (Reel and Conte 1994).

Recent study (Aldemir-Bektas 2015), on behalf of the Minnesota Department of Transportation, identified and proposed the needs and protocols for the incorporation of element-level bridges inspections to improve the efficiency and timetable of annual bridge projects. As per the latest transportation bill, the *Moving Ahead for Progress in the 21st Century Act* (MAP-

21), which emphasize the need for performance-based assessment of bridge management, Bektas investigated performance measures of MnDOT bridge inspection elements that would ensure they conform to the new AASHTO *Guide Manual for Bridge Element Inspection* methodologies. Bektas suggested that, with the expansion of the National Highway System with MAP-21, there are now bridges on the NHS for which element inspections were not required before. She also acclaimed that fundamental changes to the AASHTO *Manual for Bridge Element Inspection* (previously titled "AASHTO CoRe" elements) incuding: (1) the standardization of all condition states to four, (2) the separation of wearing surfaces and protective systems from bridge elements, and (3) the implementation of defect-based inspection methodology, may emerge issues overtime. Nonetheless, consistency amongst element inspections (by state and local inspectors) will provide quality input for bridge condition assessment, performance measures, and overall decision-making (Aldemir-Bektas 2015).

While current BMSs focus on the need for expansion to include more functional aspects of bridge performance, another study (Ikpong and Bagchi 2015) proposed procedures to include bridge resilience or vulnerability against climate impacts. The following bridge resilience indicators: abutment washout, pier scour, abutment erosion, deck flooding, and abutment permafrost stability were suggested for bridge rating purposes. According to Ikpong and Bagchi, the formulation of each indicator required capacity measurement and factor weighting to better assure investment benefits. Moreover, Ikpong and Bagchi concluded that, beyond the usual bridge condition monitoring, the rating criterion includes the basis of bridge replacement cost, failure consequence, and user cost/inconvenience (Ikpong and Bagchi 2015).

Other literatures, including (Washer, et al. 2014) presents the need for Risk-Based Inspection (RBI) practices for bridges. According to the authors, the need for RBI methodological practices differ from traditional approaches that are generally calendar based, because the setting of inspection frequencies (or intervals) and scope are not fixed or uniform (Washer, et al. 2014). RBI practices considers the effects of bridge type, age, condition, importance, environment, loading, prior problems, and other characteristics that contribute to the reliability and durability of bridges sequentially (Washer, et al. 2014). Moreover, by analyzing the likelihood of anticipated damage modes and the associated outcomes or consequences, RBI practices focus attention specifically on the damage and deterioration mechanisms that are most important for ensuring bridge safety.

2.3 Summary of Relevant Studies

Several relevant findings from previous studies regarding bridge and guardrail design, safety, and sustainability impacts are summarized below.

1) Design impact studies show that the Midwest Guardrail System (MGS), which is a modified W-beam guardrail system, is to be very efficient. Those include:

- (Reid, et al. 2011): MGS was proven to provide exceptional redirective capability in standard and special applications.
- (Thiele, et al. 2011): MGS was designed to be simple, economic (reduced bridge deck width and its associated costs), and usable both on newly constructed bridges and for retrofitting to existing bridges.
- (Rosenbaugh, et al. 2015) and (Abu-Odeh, et al. 2015): MGS was found to be crashworthy in both asphalt and concrete.

- (Stephens Jr. 2005) and (Klaiber, Wipf and Phares, et al. 2000): suggested considerations of cost-effective alternatives, particularly due to the rare-nature of crashes on LVRs.

2) Safety impact studies recognized some characteristics that tend to influence crash frequency and crash severity. Those include:

- (Mehta, et al. 2015): high presence of truck traffic were found to reduce the frequency of bridge crashes.
- (Seitz and Salfrank 2014): suggested there are benefits of bridge railings, nonetheless, policies that require installation of bridge rails on bridges between 20 ft. and 50 ft. on roads functionally classified as Local Roads with less than 50 vehicles per day need not be required.
- (Bigelow, Hans and Phares 2010) and (Turner 1984): suggested that frequency of vehicular crashes are more prevalent on bridges with smaller widths in relation to its roadway approach width.
- (Lee and Mannering 1999): suggested that, perhaps, roadside recovery space is the most important factor in reducing crash severity, in addition to road widths.
- (Hall 1982): suggested that guardrails used to protect abutments or bridges would reduce the crash severity by 50 percent.

3) Sustainability impact studies recognized that bridge condition and assessment may influence performance and overall decision-making processes. Those include:

- (Aldemir-Bektas 2015): consistency among element inspections will provide quality input for condition assessment, performance measures, and overall decision-making.
- (Jiang and Sinha 1989): life-cycle cost assessment may also be useful to determine optimal timing and efficiency of bridge activities.

Although many studies have considered bridge design and operational influences, little-to-no importance has been given to include (bridge) condition influences on the frequency and severity of bridge crashes and bridge safety. This study focused on the impacts of design (dimensional), operational, and conditional characteristics to better understand their fluencies on the frequency and severity of crashes involving bridge rail and guardrail.

CHAPTER 3 DATA COLLECTION, QUALITY, AND ASSURANCE

This chapter describes the data collection protocol and modification that was used during the analyses. Section 3.1 of this paper describes the databases and data collection methodologies. Section 3.2 explains the processing and modifications of data elements to ensure data validity.

3.1 Data Collection and QA/QC

This research required the query and integration of multiple data sources in order to explore possible relationships between characteristics of design, operational, and overall bridge proficiency. Specifically, data pertaining to bridge design measurements, traffic operation estimates, and bridge condition rating, in addition to crash record history, were considered. By means of the Iowa Department of Transportation (IowaDOT) Geographic Information Management System (GIMS) database, in addition to the IowaDOT Base Record Road and Structure Data and the Iowa Crash Data, it was possible to investigate safety performance improvements of bridge components on county roads in Iowa. A statewide query and integration of all data from 2004 to 2013 was performed using ArcGIS 10.3 software program.

3.1.1 Roadway traffic database

The Base Record Road database included all public road records in Iowa. Nonetheless, given that literature suggests greater risks on rural roads which is the scope of this research, only roadways governed under county jurisdiction were considered and collected. Nearly 87,000 miles (75%) of Iowa roadway were considered county roads, which included both multiple lanes and divided roadways. Figure 13 shows (in green) a 2010 snapshot of Iowa county roadways. The Base Record Road database included over 100 roadway elements pertaining to roadway (traffic) information and inventory. Of the county roads retained, roadway attributes such as annual average daily traffic volume of the road (AADT), speed limit of the road, roadway cross-section (e.g. width, number of lanes), and surface type (paved or unpaved), were noted and linked to corresponding bridges which were identified in Section 3.2.1 . Eighty percent of the county roadway network had unpaved surface type and twenty percent paved.

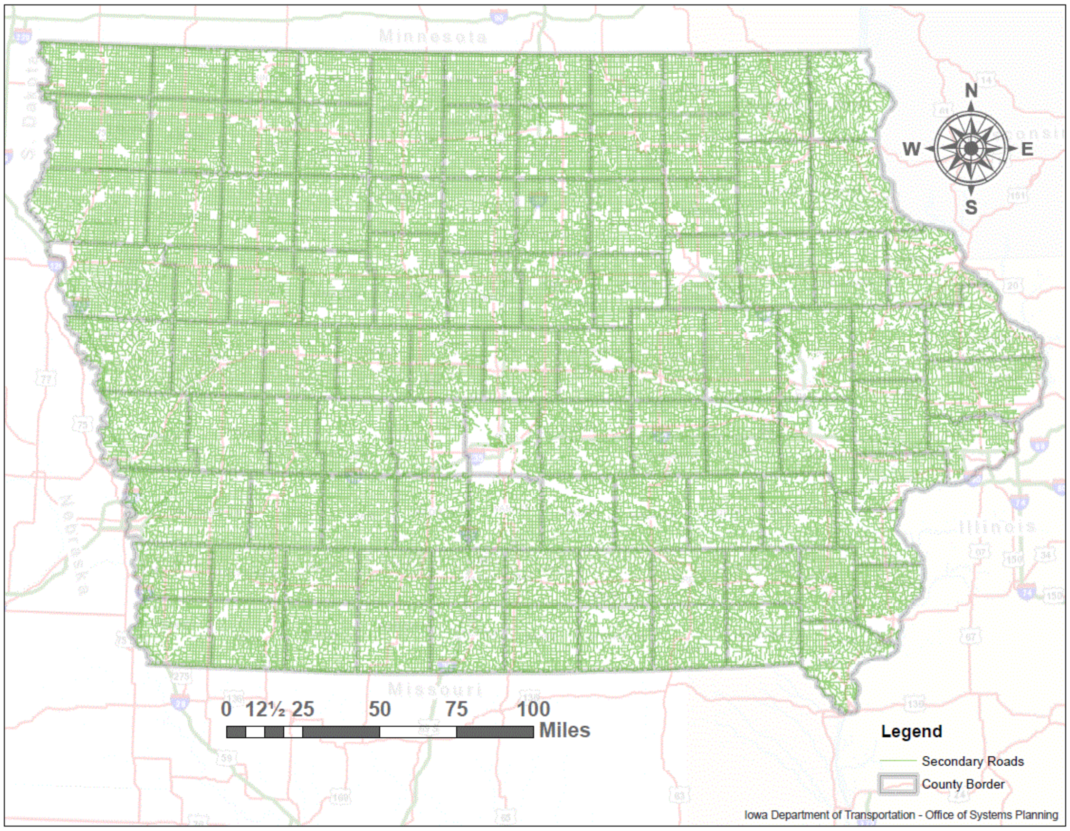


Figure 13: Snapshot of Iowa county roads in 2010 (shown in green)
Source: ArcGIS 10.3

3.1.2 Bridge structure database

The Structure database included all National Bridge Inventory (NBI) structures in the State of Iowa; specifically, those are bridges of a minimum length of 20 feet including box-culverts. For the purpose of this research, only bridges carrying vehicle traffic were considered. Figure 14 shows (in blue) location of the 18,577 bridges along county roadways which met the above criteria (2010 snapshot). The Structure Database included nearly 200 bridge elements pertaining to bridge design (including dimensional) and condition information. Characteristics such as bridge number/ID, geometry (i.e. bridge length and bridge width), construction or reconstruction year, and bridge type (i.e. concrete, timber), were extracted. Furthermore, the collected county road bridges were later divided into two groups. Group 1 were those situated on the paved road network (n = 3,970), and Group 2 were those situated on the unpaved road network (n = 14,607). Roadway characteristics from the Roadway Traffic database where extracted for each identified bridge.

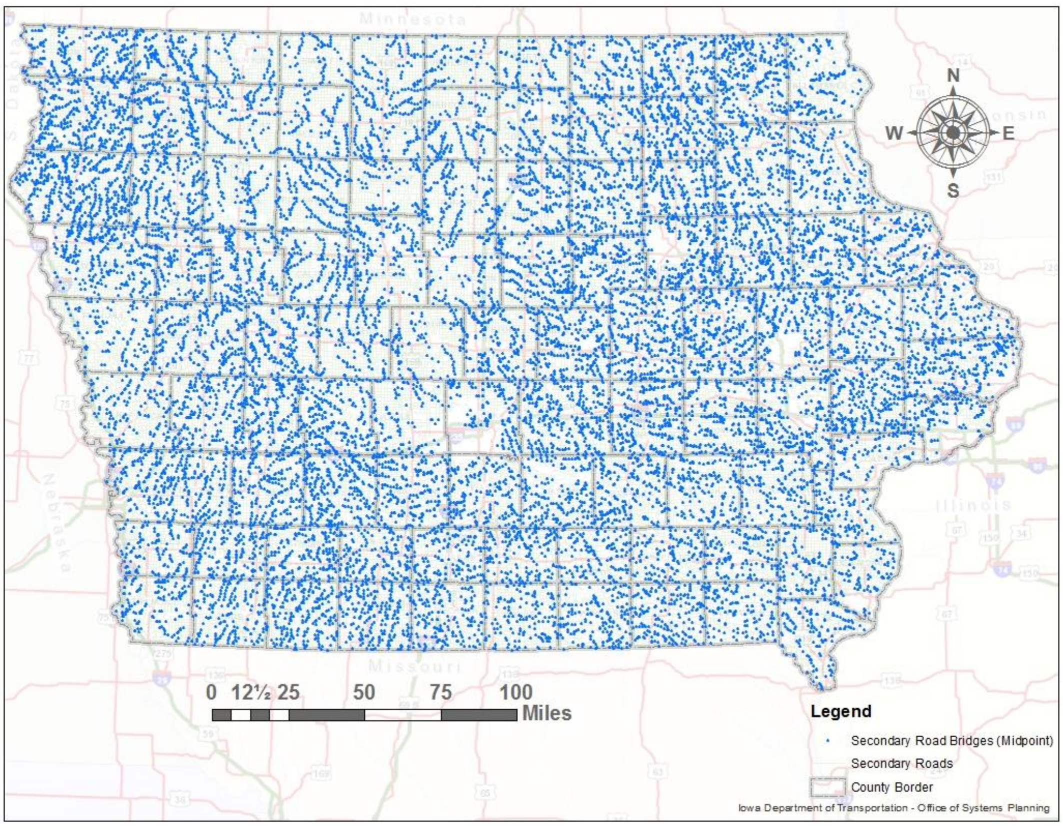


Figure 14: Snapshot of Iowa county road bridges in 2010 (shown in blue)
Source: ArcGIS 10.3

3.1.3 Vehicle crash database

Crash data from 2004 to 2013 were obtained for the roadways of interest using the Iowa Crash Database which included around 550,000 vehicle crashes. The Crash database has reported crashes of a minimum estimated property damage of $1,500. The crash database contains information at crash-, vehicle-, and person-level such as injury status/severity, number of vehicles and individuals involved/injured, sequence of events, and roadway condition (i.e. wet surface). Since the intent of the research was to investigate bridge-related crashes, only crashes coded as "collision with fixed object": bridge, bridge rail, overpass, underpass, structure support, culvert, guardrail, concrete barrier, impact attenuator, or other fixed object was noted in one of the sequence of events fields, or in the fields: "most harmful event", "first harmful event", and "fixed object struck" was considered. This represented less than 1% (nearly 3,947) of all crashes.

Since some spatial inaccuracy may be present in the crash database, crashes located 164 feet (50 meters) beyond a county bridge were excluded. Crashes meeting the following criteria were also excluded (1) occurred along ramps and other non-mainline road segments, (2) involved collision with an animal, (3) occurred along a bridge underpass based on vehicle initial direction of travel, and (4) involved multiple vehicles. Multiple vehicle crashes were excluded because those often are situations beyond the measures of roadway design, or bridge characteristics, and should be investigated independently. Specifically, the main disparity between multiple- (MVC) and single-vehicle crashes (SVC) relates to their effects on injury severity—multiple-vehicle collisions increase the probability of a severe injury, whereas single-vehicle collisions increase the probability of an injury (Cerwick 2013). Involvement of both observations within a model could produce misleading interpretation of parameters. Also, by means of visual inspection using

Google Street View and Microsoft Bird's eye additives in ArcGIS, crashes were verified based on direction of travel and other measured attributes.

After excluding crashes as noted above, 862 single-vehicle, run-off-road crashes involving vehicles striking a bridge or bridge-related component were identified and used in the successive analysis. Figure 15 shows (in red) all crashes associated to bridges along county roadways in Iowa from 2004 to 2013.

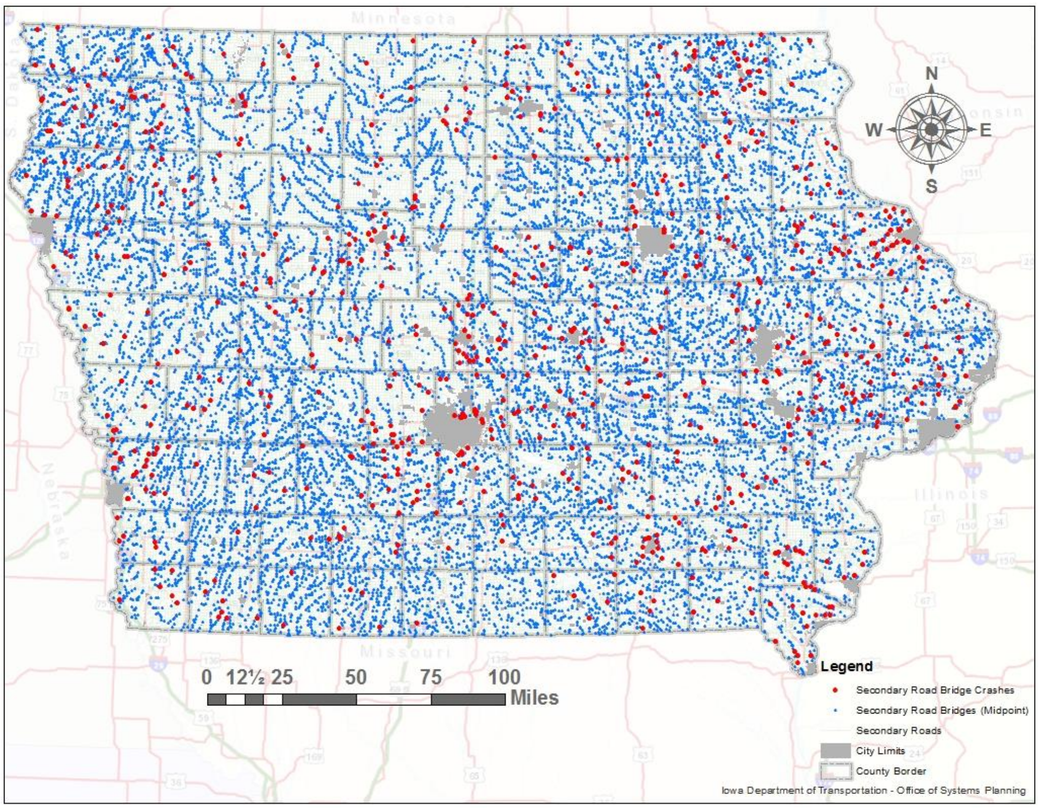


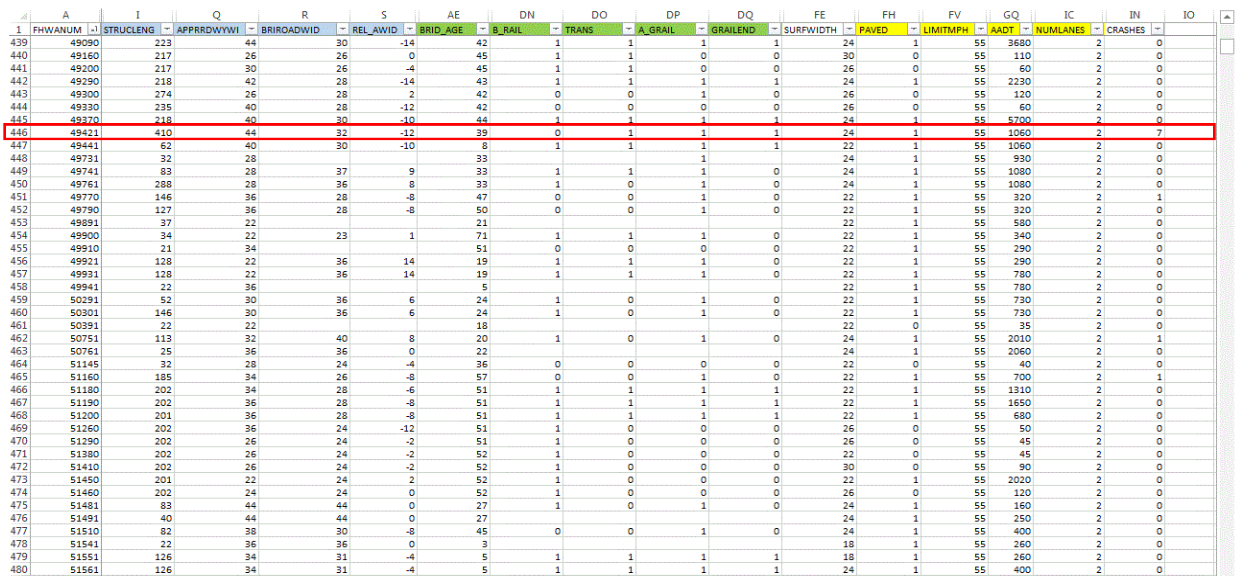
Figure 15: Iowa county road bridge crashes from 2004 to 2013 (shown in red)
Source: ArcGIS 10.3

3.2 Data Integration Methodology

The next step was to link bridge-related crashes (as described in Section 3.1.3) with corresponding bridges (as identified in Section 3.1.2). Two different analyses were conducted.

The first evaluated the relationship between crashes and bridge characteristics. The second evaluated crash severity. As a result, two different databases were developed.

The first database included bridges as the main feature of interest. Each row represented one bridge and contained the corresponding roadway operations (traffic volumes), designs (widths, lengths), and bridge condition characteristics. The number of crashes corresponding to each bridge was also included as a field. Figure 16 shows a screenshot of the frequency dataset.



	A	I	Q	R	S	AE	DN	DO	DP	DQ	FE	FH	FV	GQ	IC	IN	IO
1	FHWANUM	STRUCLENG	APPRRDWYWI	BRIROADWID	REL_AWID	BRID_AGE	B_RAIL	TRANS	A_GRAIL	GRAILEND	SURFWIDTH	PAVED	LIMITMPH	AADT	NUMLANES	CRASHES	
439	49090	223	44	30	-14	42	1	1	1	1	24	1	55	3680	2	0	
440	49160	217	26	26	0	45	1	1	0	0	30	0	55	110	2	0	
441	49200	217	30	26	-4	45	1	1	0	0	26	0	55	60	2	0	
442	49290	218	42	28	-14	43	1	1	1	1	24	1	55	2230	2	0	
443	49300	274	26	28	2	42	0	0	1	0	26	0	55	120	2	0	
444	49330	235	40	28	-12	42	0	0	0	0	26	0	55	60	2	0	
445	49370	218	40	30	-10	44	1	1	1	1	24	1	55	5700	2	0	
446	49421	410	44	32	-12	39	0	1	1	1	24	1	55	1060	2	7	
447	49441	62	40	30	-10	8	1	1	1	1	22	1	55	1060	2	0	
448	49731	32	28			33			1		24	1	55	930	2	0	
449	49741	83	28	37	9	33	1	1	1	0	24	1	55	1080	2	0	
450	49761	288	28	36	8	33	1	0	1	0	24	1	55	1080	2	0	
451	49770	146	36	28	-8	47	0	0	1	0	22	1	55	320	2	1	
452	49790	127	36	28	-8	50	0	0	1	0	22	1	55	320	2	0	
453	49891	37	22			21					22	1	55	580	2	0	
454	49900	34	22	23	1	71	1	1	1	0	22	1	55	340	2	0	
455	49910	21	34			51	0	0	0	0	22	1	55	290	2	0	
456	49921	128	22	36	14	19	1	1	1	0	22	1	55	290	2	0	
457	49931	128	22	36	14	19	1	1	1	0	22	1	55	780	2	0	
458	49941	22	36			5					22	1	55	780	2	0	
459	50291	52	30	36	6	24	1	0	1	0	22	1	55	730	2	0	
460	50301	146	30	36	6	24	1	0	1	0	22	1	55	730	2	0	
461	50391	22	22			18					22	0	55	35	2	0	
462	50751	113	32	40	8	20	1	0	1	0	24	1	55	2010	2	1	
463	50761	25	36	36	0	22					24	1	55	2060	2	0	
464	51145	32	28	24	-4	36	0	0	0	0	22	0	55	40	2	0	
465	51160	185	34	26	-8	57	0	0	1	0	22	1	55	700	2	1	
466	51180	202	34	28	-6	51	1	1	1	1	22	1	55	1310	2	0	
467	51190	202	36	28	-8	51	1	1	1	1	22	1	55	1650	2	0	
468	51200	201	36	28	-8	51	1	1	1	1	22	1	55	680	2	0	
469	51260	202	36	24	-12	51	1	0	0	0	26	0	55	50	2	0	
470	51290	202	26	24	-2	51	1	0	0	0	26	0	55	45	2	0	
471	51380	202	26	24	-2	52	1	0	0	0	22	0	55	45	2	0	
472	51410	202	26	24	-2	52	1	0	0	0	30	0	55	90	2	0	
473	51450	201	22	24	2	52	1	0	0	0	22	1	55	2020	2	0	
474	51460	202	24	24	0	52	1	0	0	0	26	0	55	120	2	0	
475	51481	83	44	44	0	27	1	0	1	0	24	1	55	160	2	0	
476	51491	40	44	44	0	27					24	1	55	250	2	0	
477	51510	82	38	30	-8	45	0	0	1	0	24	1	55	400	2	0	
478	51541	22	36	36	0	3					18	1	55	260	2	0	
479	51551	126	34	31	-4	5	1	1	1	1	18	1	55	260	2	0	
480	51561	126	34	31	-4	5	1	1	1	1	24	1	55	400	2	0	

Figure 16: Screenshot of crash frequency dataset in Excel

The second dataset was to assess the likelihood of injury for bridge-related crashes. As a result, each row represented a vehicle occupant and included the associated person, vehicle, and crash characteristics as shown in Figure 17. A crash could be represented by more than one occupant as noted by the red box in Figure 17.

	A	B	C	E	G	J	L	R	Z	AC	AH	BB	BN	CT	DI	EC	EL	EM
1	CRASH_KEY	CRASHES	FHWANUM	PERSON_KEY	INJSTATUS	OCCUPAGE	OCCUPGEN	PROTECT	DRIVERAGE	DRIVERGEN	DRUG_ALC	VEHAGE	OCCUPANT	WISS	LIGHTDAY	PROPDMG		
85	2008028812	2	45320	20080288120001	5	40	M	1	40	M	1	6	1	0	1	7200		
86	2013034581	2	45320	20130345810001	5	57	M	1	57	M	0	6	2	0	1	5500		
87	2007044398	1	45360	20070443980001	4	19	M		19	M	0	17	1	0	0	7000		
88	2008031071	1	46040	20080310710001	5	18	M	1	18	M	0	20	1	0	0	3000		
89	2005059499	1	46520	20050594990001	5	31	F		31	F	0	7	3	0	1	4815		
90	2006057004	1	46601	20060570040001	5	32	F	1	32	F	0	3	2	1	1	2200		
91	2013013852	1	47001	20130138520002	5	27	F		27	F	0	7	1	0	1	9000		
92	2012010919	1	47450	20120109190001	2	35	M	0	35	M	1	1	1	0	1	10500		
93	2006018834	7	49421	20060188340001	5	16	F	1	16	F	0	18	1	0	1	1000		
94	2008017930	7	49421	20080179300001	5	52	F	1	52	F	0	2	1	1	1	2200		
95	2008017931	7	49421	20080179310001	5	51	F	1	51	F	0	5	1	1	1	2000		
96	2008061066	7	49421	20080610660001	3	27	M	0	27	M	1	6	1	0	0	4000		
97	2009030814	7	49421	20090308140001	3	21	M	1	21	M	0	5	1	0	0	3000		
98	2011002527	7	49421	20110025270001	5	16	F		16	F	0	9	1	1	0	4200		
99	2012007821	7	49421	20120078210001	5	23	F	1	23	F	0	16	1	1	1	2500		
100	2006028311	1	49770	20060283110001	5	49	M		49	M	0	14	1	0	1	3000		
101	2008044176	1	50751	20080441760001	5	39	M		39	M	0	7	1	0	0	1500		
102	2008039393	1	51160	20080393930004	4	1	M	1	21	F	0	3	3	0	0	5000		
103	2008039393	1	51160	20080393930003	4	26	M		21	F	0	3	3	0	0	5000		
104	2008039393	1	51160	20080393930001	3	21	F		21	F	0	3	3	0	0	5000		
105	2010052585	1	51581	20100525850001	5	17	M	1	17	M	0	14	4	0	1	3500		
106	2013019849	1	52131	20130198490001	3	30		1	30		0	10	1	0	0	23000		
107	2007024315	2	52330	20070243150001	4	54	M		54	M	1	15	1	1	1	2500		
108	2007032643	2	52330	20070326430001	4	26	M		26	M	0	10	1	1	0	2750		
109	2004054104	1	53500	20040541040001	5	49	F	1	49	F	0	2	1	1	1	24000		
110	2007002048	1	53525	20070020480001	4	53	F		53	F	0	11	1	1	0	3500		
111	2013039689	1	53660	20130396890001	5	56	F	1	56	F	0	15	1	1	0	3500		
112	2011044526	1	54130	20110445260001	4	17	F	1	17	F	0	6	1	0	0	13000		
113	2005055813	1	54160	20050558130001	5	53	F	1	53	F	0	8	1	0	0	3000		
114	2013032400	1	55781	20130324000001	4	39	M	1	39	M	0	2	1	0	1	5000		
115	2008025112	1	56370	20080251120001	3	17	M	1	17	M	0	11	1	0	1	10000		
116	2004030540	1	57500	20040305400001	5	53	F	1	53	F	0	12	2	0	1	4542		
117	2009008159	1	57640	20090081590004	3	15	M	0	16	M	0	7	3	0	0	6013		
118	2010052659	1	57650	20100526590001	3	60	F	0	60	F	0	5	1	1	1	8100		
119	2005007730	1	59470	20050077300001	5	42	M		42	M	0	5	1	0	0	3380		
120	2004009030	1	59730	20040090300001	5	20	M	1	20	M	0	11	2	1	0	2500		
121	2013032226	1	61921	20130322260001	5	22	M	1	22	M	1	12	1	0	0	13700		
122	2009016905	1	63070	20090169050001	4	18	F	1	18	F	0	5	1	0	1	1800		
123	2006036442	1	64480	20060364420001	5	31	M	1	31	M	0	8	3	0	0	4000		
124	2004055594	3	65450	20040555940001	3	22	M	1	22	M	1	9	1	0	0	3000		
125	2006037180	3	65450	20060371800001	4	28	M	1	28	M	0	0	1	0	0	8000		
126	2011021559	3	65450	20110215590001	1	28	M	1	28	M	0	5	1	0	0	11000		
127	2012033714	1	65830	20120337140004	3	20	F	1	20	F	0	9	1	0		14500		
128	2006052460	1	66225	20060524600001	9	72	F	1	72	F	0	4	1	1	1	13500		

Figure 17: Screenshot of occupant injury dataset in Excel

There were three steps each in the integration procedures for the frequency and occupant injury datasets. Figure 18 shows a flow chart of the integration procedure for both the frequency and occupant injury datasets. Each of the databases are described in more detail in the following sections.

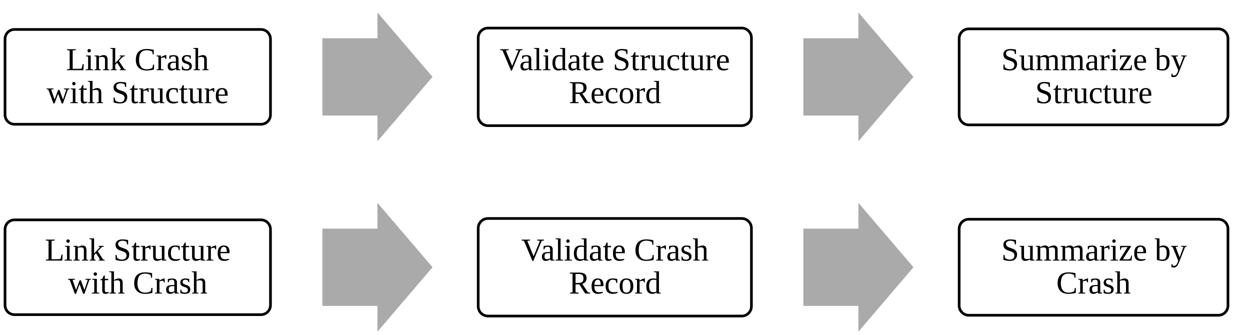


Figure 18: Integration flow charts of the frequency dataset (top) and occupant injury dataset (bottom)

3.2.1 Description of the frequency database

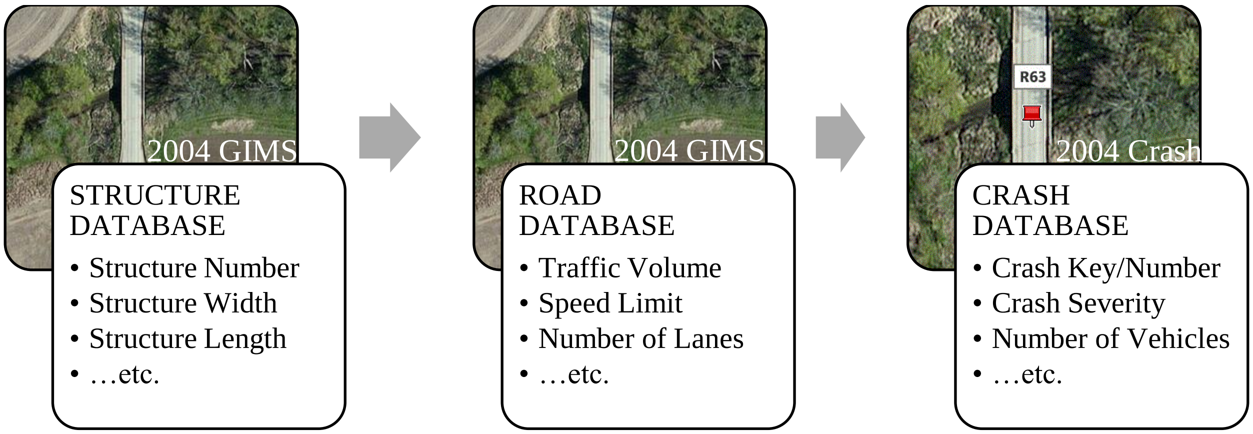


Figure 19: Structure, roadway, and crash data linkage in ArcGIS

When a bridge experienced multiple crashes (either in same year or different years) within the analysis period, the bridge and roadway attributes for the various years were reviewed to check for differences in values between years. When differences were noted (i.e. if AADT fluctuate from year to year), the weighted average was used.

In some cases, a bridge had no associated crashes over the 10-year period. Crash frequency was recorded as zero and roadway, traffic operation, and bridge data were extracted for 2009.

Figure 20 illustrates bridge-associated crashes summarized by bridge. A total of 862 vehicle-crashes were noted which involved 735 bridges and 1,554 occupants. The proceeding figure (Figure 21) show the distribution of bridges by crash count. The distribution shows 17,842 bridges experiencing zero crash; 638 bridges experiencing one crash each, 75 bridges experiencing two crashes each; 17 bridges experiencing three crashes each; four bridges experiencing four crashes each; zero bridges experiencing five or six crashes; and one bridge experiencing up to seven crashes in the 10-year period (see Figure 22).

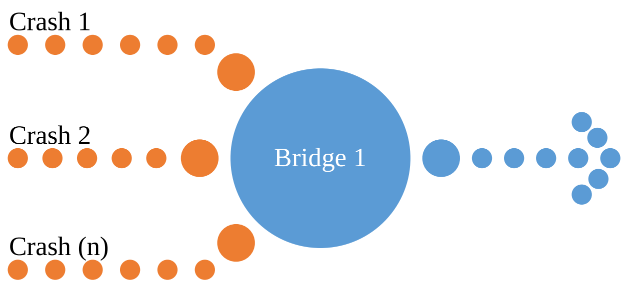


Figure 20: Bridge-associated crashes summarized by bridges in the frequency dataset

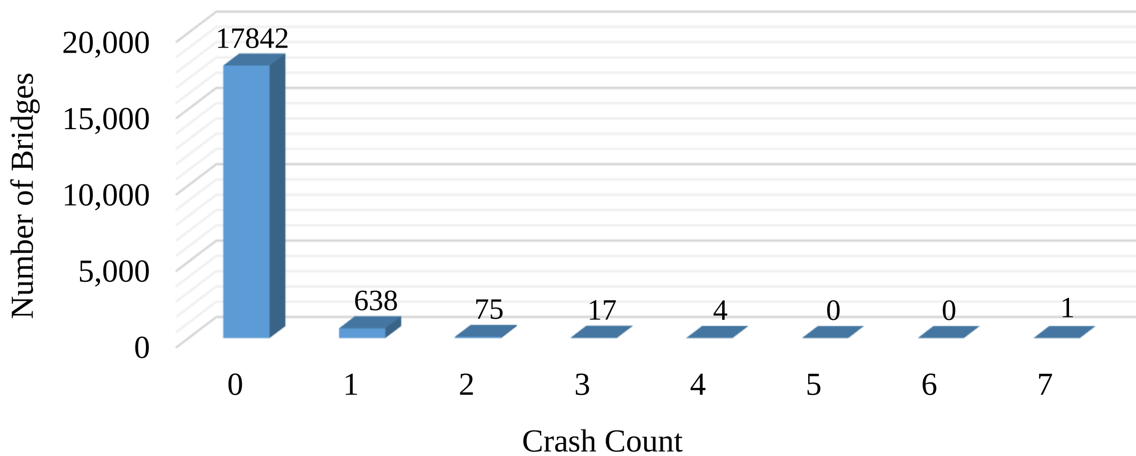


Figure 21: The distribution of bridges by crash count

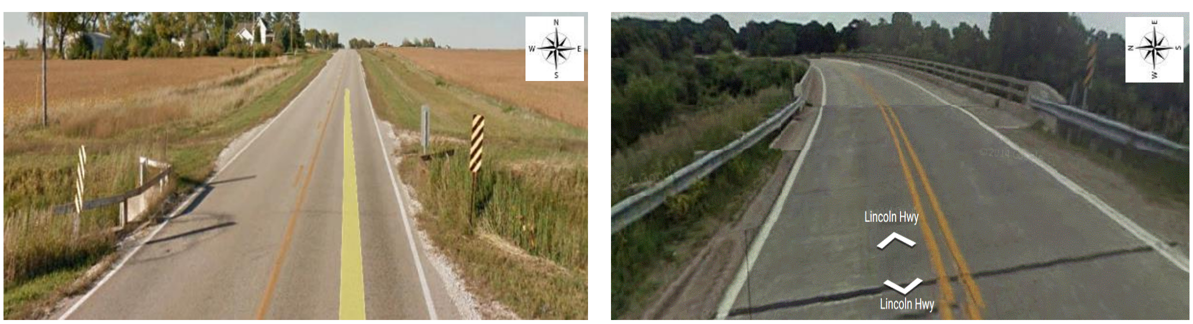


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Figure 22: Examples of bridges struck multiple times
Source: ArcGIS 10.3

3.2.2 The occupant injury database

The first step to develop the occupant injury database was to ensure that each bridge-related crash was associated with a bridge. As noted above, each row in the database represented one occupant involved in a bridge crashes. Characteristics including injury status, occupant characteristics (e.g. age, vehicle (type, occupants), crash circumstance (environment condition), and bridge characteristics were extracted for each occupant as was illustrated in Figure 17 (see Section 3.1). Figure 23 show the distribution of occupant severity for bridge-related crashes.

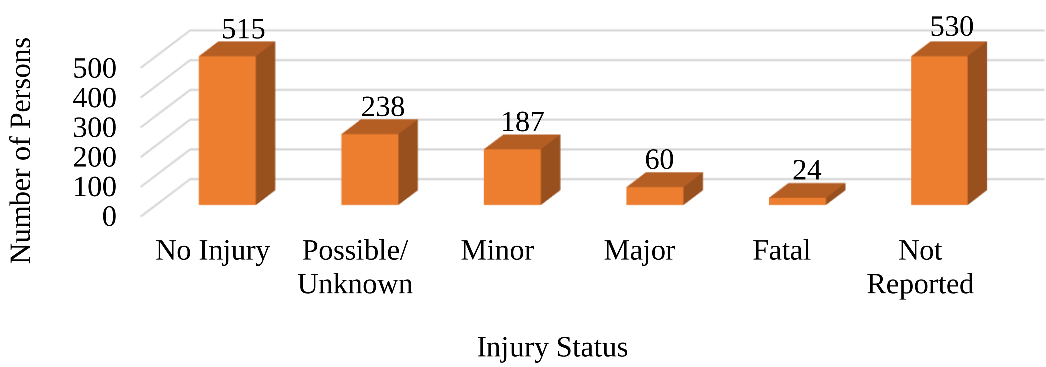


Figure 23: The distribution of vehicle occupants by injury status

CHAPTER 4 CRASH FREQUENCY ANALYSIS

This chapter investigated parameters associated with the frequency of crashes involving county road bridges. Furthermore, statistical regression analysis was built to test the significance of parameters and patterns found within the crash frequency dataset. Section 4.1 introduces its statistics. Section 4.2 presents the analysis framework, and Section 4.3 presents the results.

4.1 Crash Frequency Data

Within the crash frequency data analysis, three classes of variables were used to ascertain the associative properties of roadway and bridge characteristics on the frequency of crashes involving bridges and guardrails. Those classes of variables included: (roadway) traffic operations variables, bridge design variables, and bridge condition variables as shown in Table 9. Roadway traffic operations variables indicated parameters pertaining to the traffic movement and exposure rate of bridges, including traffic volume, truck percentage, speed, etc. Bridge variables included bridge length, width, type, etc. Bridge condition variables indicated parameters pertaining to the status-quo of bridges, including bridge age, and condition ratings.

Table 9: The three (3) classes of variables considered within the frequency dataset

Roadway (Traffic) Operations Variables (TO)	Bridge Design Variables (BD)	Bridge Condition Variables (BC)
Traffic Volume (AADT)	Bridge Length	Bridge Age*
Truck Percentage*	(Approach) Bridge Width	Superstructure Condition
Speed Limit	Bridge Roadway Width	Structure Condition
Number of Lanes	Relative Bridge Width*	Deck Condition
Roadway Surface Width	Bridge Type	Alignment Condition
Roadway Lane Width*	Bridge Flare	
Local Road	Shoulder Width	
Farm-to-Market Road	Median Width	
	Terrain	
	Paved/Unpaved Road	

As shown in the table, several variables were derived from existing characteristics to characterize bridge performance. Those variables are followed by an asterisk. For instance, the variable REL_AWID* was derived from calculating the algebraic difference between the bridge and approach roadway widths.

Figure 24 shows an example of a typical roadway bridge cross-section with respect to its approach, surface, and bridge widths. Relative bridge width of negative value indicates bridges that are narrower than the traveled way. Also, both the bridge and approach widths include shoulder and median widths which is captured within the relative bridge width.

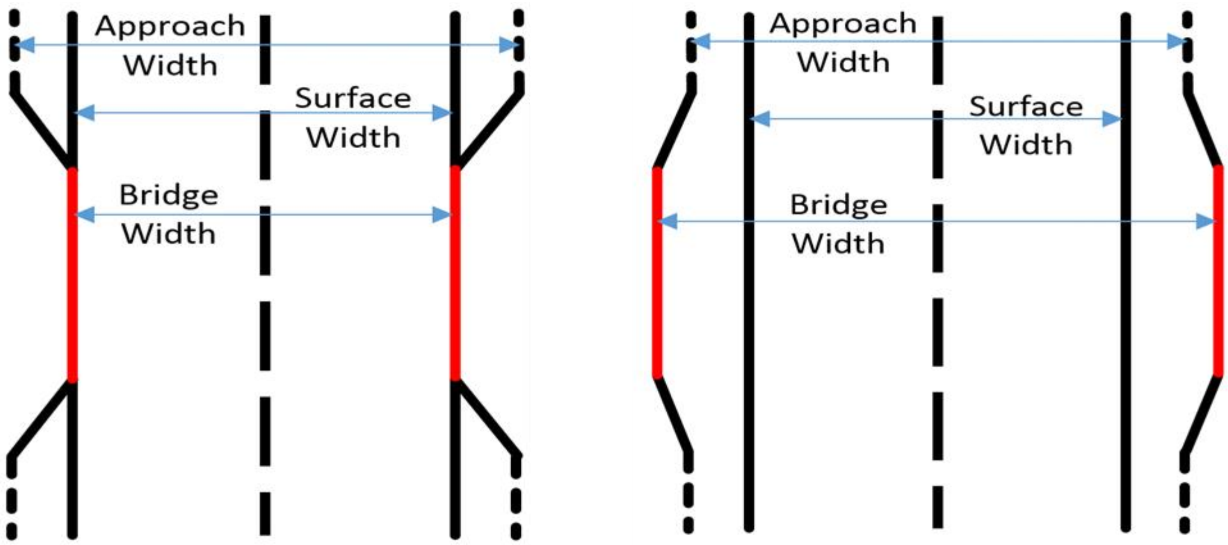


Figure 24: Examples of typical cross-sections with a negative relative bridge width (left) and a positive relative bridge width (right)

The proceeding tables (Table 10, Table 11, and Table 12) present the descriptive and summary statistics of the TO, BD, and BC variables, respectively. The descriptive statistics include the description, and the summary statistics include the mean, range (min/max), and standard deviation of each variable considered. Furthermore, prior to the use of each variable, Pearson's correlation analysis was performed to check for multicollinearity within the analysis.

The Pearson’s correlation analysis results can be found in Figure 30 through Figure 32 of appendix B. In all, results of multicollinearity amongst parameters used ranged from -0.144 to 0.386. Variables conveying strong correlations were considered and not included simultaneously.

Table 10: The descriptive and summary statistics of TO variables considered

Variable	Description	Min	Mean	Max	Std. Dev.
AADT	The annual average number of vehicle traffic traversing a bridge daily.	5	176.15	8900	408
SU_TRUCK*	The percentage of AADT that is single-unit trucks.	0.00	0.055	0.95	0.094
MU_TRUCK*	The percentage of AADT that is multi-unit trucks.	0.00	0.022	0.85	0.047
LIMITMPH	The speed limit of roadway traversing a bridge.	10	54.52	55	3.30
NUMLANES	The number of lanes traversing a bridge.	1	2.00	4	0.034
SURFWID	The roadway surface width of a bridge.	10	23.11	42	2.517
LANEWID*	The roadway lane width of a bridge.	10	11.56	14	1.254
LOCAL_RD	An indicator for a bridge on local roadway: 1 = local road; 0 otherwise	0	0.626	1	0.484
PAVED	An indicator for a bridge on paved roadway: 1 = paved; 0 otherwise	0	0.214	1	0.410

Table 11: The descriptive and summary statistics of BD variables considered

Variable	Description	Min	Mean	Max	Std. Dev.
STRUCLEN	The length of a bridge in feet.	18	78.4	1265	75.634
APPRRDWY	The normal width of usable roadway approaching the bridge measured in feet, including both shoulders, roadways, and median.	9	26.88	99	5.166
BRIROADW	The most restrictive minimum curb-to-curb distance between the bridge railings measured in feet.	12	23.77	100	5.217
REL_AWID*	The algebraic difference between the bridge and approach roadway widths, measured in feet.	-22	-3.02	38	4.736
CONCRETE	An indicator for a concrete bridge type : 1 = concrete; 0 otherwise	0	0.509	1	0.500
STEEL	An indicator for a steel bridge type: 1 = steel; 0 otherwise	0	0.373	1	0.484
TIMBER	An indicator for a timber bridge type: 1 = timber; 0 otherwise	0	0.118	1	0.323
CULVERT	An indicator for a culvert bridge type: 1 = culvert; 0 otherwise	0	0.158	1	0.365
FLARE	An indicator for a bridge that is flared: 1 = flared; 0 otherwise	0	0.002	1	0.043
TOTSHDWD*	The total width of the left and right shoulder of roadway, measured in feet.	0	3.642	30	3.811
MEDWIDTH	The width of the median between the edges of traffic lanes, measured in feet.	0	0.000	0	0.000

FLAT	An indicator for the type of terrain located on both sides of the road: 1 = flat; 0 otherwise	0	0.291	1	0.454
ROLLING	An indicator for the type of terrain located on both sides of the road: 1 = rolling; 0 otherwise	0	0.655	1	0.475
HILLY	An indicator for the type of terrain located on both sides of the road: 1 = hilly; 0 otherwise	0	0.054	1	0.227

Table 12: The descriptive and summary statistics of BC variables considered

Variable	**Description**	**Min**	**Mean**	**Max**	**Std. Dev.**
BRID_AGE*	The age of a bridge prior to crash, in years.	0	37.340	138	25.280
SUP_POOR	An indicator for a bridge members physical condition: 1 = poor; 0 otherwise	0	0.072	1	0.259
SUP_OK	An indicator for a bridge members physical condition: 1 = satisfactory; 0 otherwise	0	0.444	1	0.497
SUP_GOOD	An indicator for a bridge members physical condition: 1 = good; 0 otherwise	0	0.483	1	0.500
STRUPOOR	An indicator for a bridge overall condition: 1 = poor; 0 otherwise	0	0.127	1	0.333
STRUOK	An indicator for a bridge overall condition: 1 = satisfactory; 0 otherwise	0	0.587	1	0.492
STRUGOOD	An indicator for a bridge overall condition: 1 = good; 0 otherwise	0	0.286	1	0.452
DECKPOOR	An indicator for a bridge overall deck condition: 1 = poor; 0 otherwise	0	0.082	1	0.274
DECKOK	An indicator for a bridge overall deck condition: 1 = satisfactory; 0 otherwise	0	0.467	1	0.499
DECKGOOD	An indicator for a bridge overall deck condition: 1 = good; 0 otherwise	0	0.451	1	0.498
GEO_POOR	An indicator for a bridge deck geometry condition: 1 = poor; 0 otherwise	0	0.095	1	0.293
GEO_OK	An indicator for a bridge deck geometry condition: 1 = satisfactory; 0 otherwise	0	0.741	1	0.438
GEO_GOOD	An indicator for a bridge deck geometry condition: 1 = good; 0 otherwise	0	0.165	1	0.371
ROADPOOR	An indicator for a bridge approach roadway alignment condition: 1 = poor; 0 otherwise	0	0.042	1	0.109
ROADOK	An indicator for a bridge approach roadway alignment condition: 1 = satisfactory; 0 otherwise	0	0.303	1	0.471
ROADGOOD	An indicator for a bridge approach roadway alignment condition: 1 = good; 0 otherwise	0	0.655	1	0.475
B_RAIL	An indicator for a bridge rail meeting current acceptable standard: 1 = meet standard; 0 otherwise	0	0.262	1	0.440
TRANS	An indicator for a bridge transitions meeting current acceptable standard: 1 = meet standard; 0 otherwise	0	0.209	1	0.407
A_GRAIL	An indicator for a bridge approach guardrail meeting current acceptable standard: 1 = meet standard; 0 otherwise	0	0.236	1	0.425
GRAILEND	An indicator for a bridge guardrail end meeting current acceptable standard: 1 = meet standard; 0 otherwise	0	0.171	1	0.376

Based on past literatures, a few characteristics stand out to be significant contributing factors influencing bridge crashes. Those were initially considered for the proceeding analysis. Those included characteristics such as: traffic volume and percentage of heavy vehicles, roadway cross-section features factors such as lane/shoulder widths and bridge length, roadway alignment factors such as the presence of horizontal/vertical curvature, and weather conditions factors such as the presence of rain/snow or low visibility settings.

However, due to lack of data availability and complexity of attaining, not all the aforementioned variables could be used as explanatory variables in the models. For example, characteristics such as roadway alignment factors involving curvatures were also excluded due to lack of their availability in the databases. Nevertheless, other variables (i.e. alignment condition) were considered as appropriate substitutes.

Other precautions regarding the usefulness of variables included the variability of observed measurements as shown in Table 10, Table 11, and Table 12. Bridge roadway speed limit and number of lanes, for instance, were variables removed from final consideration because they were found to be not useful to the results. Of the 18,577 bridges, merely 480 (2.6%) were serving roadways under the speed limit of 55 miles per hour. Additionally, majority (99.9%) of all bridges had 2-lane roadways. Considering the distribution of bridge width with respect to its approach, less than 59% of all county roadway bridges had narrower (negative) relative bridge widths. Also, those bridges with negative relative widths accounted for nearly 70% of all bridge crashes. In all observations, these were county road bridges with no median presence; nonetheless, less than 4% were divided ‘twin’ bridges traversing the same obstacle.

Considering the functional relationship between traffic flow (volume) and crash expectancy, several literature, including (Ardekani, Hauer and Jamei 1992), proposed a positive relationship: as traffic increase, so does crash expectancy. Specifically, though this relationship appears linear and proportionate, it is only valid at low-volume flows as assumed in this study. Traffic volume variable coefficient in this case was assumed to be a fixed parameter. On the contrary, with regard to the bridge length variable coefficient, in this case was assumed to be continuous; though as an offset parameter would indicate a linear and proportionate exposure rate as bridge length increase.

4.2 Crash Frequency Methodology

To apply the most appropriate method(s) that best reflect the data, a number of sampling approaches were considered for various statistical approaches to produce effective results. According to Washington, Karlaftis and Mannering (2011), count data can be properly modeled using a number of methods; nonetheless, the two most popular are Poisson and negative binomial regression models Count data analysis consists of nonnegative integer values and are encountered frequently in the modeling of transportation-related phenomenon. Examples of such data variables in relation to the frequency dataset include the number of crashes experienced over a period of time. Among those suitable, a negative binomial regression was selected for the crash frequency analysis given the distribution of crashes per bridge (see Figure 21).

Specifically, two fundamental assumptions when modeling Poisson regression are that the count variable should have a Poisson distribution and that the variable mean count process should equals its variance (Washington, Karlaftis and Mannering 2011). When these assumptions are violated, the variable is said to be under- or overdispersed; meaning, the variance of the number of crashes observed was statistically different from its mean count. To adjust for such violation, count data variables can be successfully modeled using negative binomial modeling. The frequency dataset was observed to violate such assumption. Thus justifying the use of negative binomial regression.

A negative binomial regression is the most widely used Poisson distribution model which accounts for under- or overdispersion (Washington, Karlaftis and Mannering 2011). Additionally, negative binomial regression variance term includes a dispersion parameter vector α that is statistically different from zero; so selection between negative binomial regression and Poisson regression models depend solely on the significance of the overdispersion parameter. Equation (3) shows the expected number of bridge crash events (y_i) per bridge (i) per period of time using a negative binomial regression.

$$E[y_i] = \exp\left(\beta_0 + \sum_{i=1}^{n} \beta_i X_i + \varepsilon_i\right) \quad or \quad \ln(E[y_i]) = \beta_0 + \sum_{i=1}^{n} \beta_i X_i + \varepsilon_i \qquad \text{Equation (3)}$$

where: $E[y_i]$ = the expected crash frequency per bridge (i) in 10 years,
β_0 = the intercept term,
β_i = the (estimated) parameter coefficient per variable X,
X_i = the explanatory variables or parameters in the model, and
ε_i = the disturbance term.

The (gamma-distributed) disturbance term ε_i has the mean of 1 and variance of α. The addition of this term allows the variance of the distribution to differ from the mean within a negative binomial regression as shown in equation (4).

$$VAR[y_i] = \mathrm{E}[y_i]\big[1 + \alpha E[y_i]\big] = E[y_i] + \alpha E[y_i]^2 \qquad \text{Equation (4)}$$

There are numerous goodness-of-fit statistics used to assess the overall fit of regression model results. Nonetheless, when selecting among alternative regressions, goodness-of-fit statistics should be considered along with model plausibility and expectations agreement (Washington, Karlaftis and Mannering 2011). The coefficient of determination (R^2) is a commonly used fundamental statistic. It serves as a numerical value ranging from zero to one which summarizes the overall strength of the model, with zero indicating a model with no predictive power and one indicating a model with perfect predictive power (Hu, Shao and Palta 2006). This statistic can be interpreted as a proportion of the variance that can be predicted (explained) given a set of explanatory/ independent variables within a model (compared to its constant-only model).

An equivalent measurement of R^2 in linear regressions is not appropriate for Poisson regressions due to the nonlinearity of the conditional means ($E[y|X]$) and heteroscedasticity (unequal variances) in the regression. For such nonlinear regressions (including negative binomial models), numerous statistics (entropy-based or variance-based), including pseudo-R^2 and McFadden R^2, can be used to summarize its predictive strength (Hu, Shao and Palta 2006). The likelihood ratio test is one common test used to assess two competing nonlinear models. It provides evidence in support of one model, usually a full or complete model, over another competing model that is restricted by having a reduced number of regressors/parameters (Washington, Karlaftis and Mannering 2011). For such analysis, the McFadden pseudo-R^2 (written as ρ^2) is utilized as the preferred statistic and is calculated using the following function:

$$\rho^2 = 1 - \frac{LL(\beta)}{LL(0)} \qquad \text{Equation (5)}$$

where $LL(\beta)$ represents the maximum log likelihood function estimate at convergence (of the finalized “restricted” model) with coefficient vector β, and $LL(0)$ represents the maximum log likelihood function estimate for its constant-only “unrestricted” model (with all parameters set at zero) (Washington, Karlaftis and Mannering 2011). Similar to linear regressions, a perfect nonlinear regression model also have a test-statistic equal to one. This implies that all selected parameters of the model are predictive (explained) with probability of one.

4.3 Crash Frequency Results

Prior to the regression analysis, thematic analysis was performed to investigate patterns and clusters of roadway and bridge variables used in the crash frequency analysis. According to Braun and Clarke, thematic analysis refers to a qualitative analytic method for identifying and analyzing patterns (themes) within data (Braun and Clarke 2006). This analysis was initially found useful to get a general understanding of how roadway and bridge variables would/should perform. This also helped reduce and remove possibly inconsistent or redundant variables from consideration.

By means of variables classification, characteristics were first classified into relatively homogeneous clusters and later modeled using regression analysis. Groupings of each variable clusters for the crash frequency analysis are shown in Table 20 of appendix B.

The crash frequency regression focused on characteristics of design, operation, and condition of bridges influencing the frequency of crashes with bridges on county roadways. The regression analysis was conducted using NLOGIT 5 statistical software.

Table 13 shows the negative binomial regression results of bridge crash frequency on Iowa’s county road network. The results suggested three operational, six design, and four conditional characteristics that are statistically correlated with the expectancy of crashes involving bridges on county roadways. In order of appearance, those included traffic volume, truck percentage, speed limit, bridge length, bridge relative width, paved surface type, bridge type timber, bridge type steel, hilly terrain, bridge age, poor bridge rail condition, poor superstructure condition, and poor road alignment condition.

Table 13: The negative binomial regression results of bridge crash frequency

Explanatory Variable	Variable Classification	Coefficient	test-statistic	Average Marginal Effect
LN_AADT	Traffic Operations	1.0	(Fixed Parameter)	0.05342 ***
MU_TRUCK	Traffic Operations	-1.00022	-1.15	-0.05343
LN_LENGTH	Bridge Design	0.21001	3.51 ***	0.01122 ***
REL_AWID	Bridge Design	-0.02603	-3.10 ***	-0.00139
PAVED	Bridge Design	-0.84581	-8.63 ***	-0.05440
TIMBER	Bridge Design	0.45012	2.71 ***	0.02961
STEEL	Bridge Design	0.17632	1.78 *	0.00957
HILLY	Bridge Design	0.25236	1.65 *	0.01508
BRID_AGE	Bridge Condition	0.00597	3.03 ***	0.00032
B_RAIL	Bridge Condition	0.08680	1.05	0.00466
SUP_POOR	Bridge Condition	0.38659	2.76 ***	0.02429
ROADPOOR	Bridge Condition	1.27677	6.09 ***	0.13395 *
Constant		-8.92975	-30.49 ***	
Overdispersion (α)		1.58486	6.39 ***	
Number of Observations		14,822		
Log-likelihood at zero		-3609.31803		
Log-likelihood at convergence		-2529.07291		
Goodness-of-Fit (ρ^2)		0.299293		

Note: ***, **, * ==> Significance at 1%, 5%, 10% level.

As described in the descriptive statistics, majority of the explanatory variables included in the regression model were indicator (dummy) variables. Those variables are highlighted in yellow. More importantly, the associated test-statistic (Student's t-statistic) indicates that some characteristics has significantly stronger explanatory properties than others, as indicated by the quantity of asterisks. Also, as evident by the significance of the overdispersion parameter, the negative binomial model results confirms to be statistically different from its Poisson model results. Based on the model results, major findings in regard to bridge crash frequency are as follows.

1) It can be observed that traffic volume has a significant effect on crashes. Though treated as a fixed parameter in the model, its marginal effect shows that a one unit increase in traffic volume would increase the expected mean of bridge crash frequency by 0.05342.
2) A one unit increase in the percentage of truck traffic would decrease the expected bridge crash frequency by 0.05343. However, it should be noted that the data represented little-to-no truck traffic, therefore its significance in the model is not prevalent.
3) Bridge length variable showed to play a significant role on bridge crash frequency. As a continuous parameter, a one foot increase in the length of a bridge would increase the expected bridge crash frequency by 0.01122.
4) The relative width of a bridge (with respect to its approach width) was observed to have an overall significant effect on crashes, however, its marginal effect was not prevailing. Specifically, a one foot increase in the relative width of a bridge (from narrow to wide) would decrease the expected bridge crash frequency by 0.00139. This width also accounts for the presence of shoulders and other roadway widths.
5) The pavement indicator variable was observed to have an overall significant effect on crashes. Nonetheless, a change from unpaved to paved bridge surface type would decrease the expected bridge crash frequency by 0.05440.
6) The indicator variables for bridge type had overall significant effects on crashes. Nonetheless, a change (from concrete) to timber or steel bridge type would increase the expected bridge crash frequency by 0.02961 and 0.00957 respectively. Specifically, timber bridges presents the greatest risks as supposed to steel or concrete, and accounted for merely 12% of all county road bridges in Iowa.

7) The roadside terrain indicator variable was observed to have a significant effect on crashes. Nonetheless, a change in roadside terrain from flat or rolling to hilly would increase the expected bridge crash frequency by 0.01508. Although, less than 6% of bridges had hilly terrains; as supposed to flat (30%) and rolling (64%).
8) The age of a bridge was observed to have a significant overall effect on crashes, however, its marginal effect was not prevailing. A one year increase in the age of a bridge would increase the expected bridge crash frequency by 0.00032.
9) Lastly, the indicator variables (railing, superstructure, and approach roadway alignment) for bridge conditions were observed to have overall significant effects on crashes. A change in bridge (railing, superstructure, or approach roadway alignment profile) condition from good or satisfactory to poor would increase the expected bridge crash frequency by 0.00466, 0.02429, and 0.13395 respectively. Specifically, poor approach roadway alignment profile presents the greatest risk of a bridge crash. Less than 5% of bridges were observed to have inadequate or poor road alignment; nonetheless, those characterized as having poor road alignment accounted for merely 13% of bridge crashes.

Overall, the goodness-of-fit statistic ρ^2 of 0.299 for the bridge crash frequency regression model suggests the appropriate overall fit of the frequency dataset results given its Poisson distribution. Specifically, the inclusion of the dispersion parameter in the crash frequency results suggest evidence in support of a negative binomial regression in contrast to a Poisson regression. Other variations of NB regressions, including random-parameters negative binomial models, showed to be slightly better but not necessary.

CHAPTER 5 INJURY SEVERITY ANALYSIS

This chapter investigated parameters associated with the severity of injuries sustained by vehicle occupants from crashes involving county road bridges. Furthermore, statistical regression analysis was built to test the significance of parameters and patterns found within the occupant injury severity dataset. Section 4.1 introduces its statistics. Section 4.2 presents the analysis framework, and Section 4.3 presents the results.

5.1 Injury Severity Data

Within the occupant injury data analysis, three classes of variables were used to ascertain properties of person, vehicle, and crash characteristics influencing the outcome of occupant injury status of crashes involving bridges and guardrails. Those classes of variables included: (occupant) person-level variables, vehicle-level variables, and crash-level variables as shown in Table 14. Person-level variables indicated parameters pertaining to the individual involved in a crash, including occupant age, gender, protection, etc. Vehicle-level variables indicated parameters pertaining to the vehicle involved, including vehicle configuration, age, driver, etc. Lastly, crash-level variables indicated parameters pertaining to the crash circumstance, including the surface and lighting condition at time of crash, drug- or alcohol-related incident, etc.

Table 14: The three (3) classes of variables considered within the occupant injury dataset

Person-Level Variables (PL)	**Vehicle-Level Variables (VL)**	**Crash-Level Variables (CL)**
Occupant Age	Vehicle Configuration	Surface Condition*
Occupant Gender	Vehicle Age*	Lighting Condition*
Occupant Protection	Driver Age	Drug/Alcohol-related
Seating Position	Driver Gender	Work zone-related
Ejection	Number of Occupants	Estimated Property Damages
Airbag Deployment	Vehicle Point of Impact	

As shown in the table, several new variables were also derived to characterize injury status outcome. Those variables are followed by an asterisk. The proceeding tables (Table 15, Table 16, and Table 17) present the descriptive and summary statistics of the PL, VL, and CL variables, respectively. The Pearson's correlation analysis results of these variables can also be found in Figure 33 through Figure 35 of appendix B.

Table 15: The descriptive and summary statistics of PL variables considered

Variable	**Description**	**Min**	**Mean**	**Max**	**Std. Dev.**
OCCU_AGE	The age of occupant, in years	0	32.135	88	16.477
OCCU_GEN	An indicator for an occupant gender: 1 = female; 0 otherwise	0	0.357	1	0.479
PROTECT	An indicator for an occupant protection: 1 = wearing a protection; 0 otherwise	0	0.881	1	0.324
FRNTSEAT	An indicator for an occupant seating position: 1 = seated in front seat; 0 otherwise	0	0.962	1	0.192
EJECT	An indicator for an occupant ejected prior to crash: 1 = ejected; 0 otherwise	0	0.022	1	0.146
AIRBAG	An indicator for an occupant airbag deployment: 1 = airbag deployed; 0 otherwise	0	0.382	1	0.486

Table 16: The descriptive and summary statistics of VL variables considered

Variable	**Description**	**Min**	**Mean**	**Max**	**Std. Dev.**
CAR	An indicator for vehicle configuration: 1 = passenger car, van, or SUV; 0 otherwise	0	0.959	1	0.198
TRUCK	An indicator for vehicle configuration: 1 = single or multi-unit truck; 0 otherwise	0	0.021	1	0.145
M_CYCLE	An indicator for vehicle configuration: 1 = motorcycle; 0 otherwise	0	0.002	1	0.044
VEH_AGE	The age of vehicle prior to crash, in years	0	9.868	44	6.243
DRI_AGE	The age of driver of vehicle, in years	14	32.395	88	16.146
DRI_GEN	An indicator for a driver gender: 1 = female; 0 otherwise	0	0.341	1	0.474
OCCUPANT	The number of occupants in vehicles	1	1.518	6	0.956
IMPFRONT	An indicator for a vehicle point initial of impact: 1 = front of vehicle; 0 otherwise	0	0.832	1	0.374
IMPSIDE	An indicator for a vehicle point initial of impact: 1 = side of vehicle; 0 otherwise	0	0.082	1	0.275
IMPREAR	An indicator for a vehicle point initial of impact: 1 = rear of vehicle; 0 otherwise	0	0.070	1	0.255

Table 17: The descriptive and summary statistics of CL variables considered

Variable	Description	Min	Mean	Max	Std. Dev.
WISS	An indicator for a crash surface condition: 1 = wet, icy, snowy, or slushy condition; 0 otherwise	0	0.339	1	0.474
LIGHTDAY	An indicator for a crash environment visibility: 1 = daylight condition; 0 otherwise	0	0.518	1	0.500
DRUG_ALC	An indicator for a crash involving drug or alcohol: 1 = drug or alcohol related; 0 otherwise	0	0.145	1	0.352
WZRELATE	An indicator for a crash involving work zone area: 1 = work zone related; 0 otherwise	0	0.007	1	0.082
TPROPDMG	The estimated amount of property damage, including non-vehicular, in thousands of dollars	0	7.986	310	14.866

In all, some variables were excluded from the final statistical models due to either multicollinearity issues or (statistic) distribution assumption violations. Detailed illustrations of the distribution of all variables used can also be found in Figure 30 through Figure 35 of appendix B.

5.2 Injury Severity Methodology

An ordered probabilistic regression model (probit or logit) was considered appropriate for the occupant injury dataset given the ordinal-scale of occupant injury status distribution (see Figure 23). Specifically, given such cross-sectional data when a single crash is potentially observed repeatedly, it is important to account for correlation amongst crashes involving multiple occupants. In such cases, the use of random-effects model is appropriate. These type of models have been widely used since the mid-1970s ((Washington, Karlaftis and Mannering 2011) and (McKelvey and Zavoina 1975)). Precisely, ordered models are derived by defining an unobserved variable z that is used as a basis for modeling the ordinal ranking of occupant injury data in this case. This unobserved variable is typically specified as a linear function for each observation. Equation (6) shows the specified (z) function, defined as an unobserved latent variable used for the basis of modeling each observed ordinal injury outcome of a crash event (y) with ε random disturbance.

$$z = \beta X + \varepsilon \qquad \text{Equation (6)}$$

where: $z =$ a latent variable used for the basis of modeling observed ordinal injury outcome,
$\beta =$ the (estimated) parameter coefficient per variable X,
$X =$ the explanatory variables or parameters in the model, and
$\varepsilon =$ the disturbance term.

The proceeding figure (Figure 25) shows an illustration of an ordered probability parameter thresholds using equation (6) for the observed ordinal occupant injury dataset (y) per occupant defined as:

$$\begin{array}{lll} y = 5 & \text{if} & z > \mu_3 \\ y = 4 & \text{if} & \mu_2 < z \leq \mu_3 \\ y = 3 & \text{if} & \mu_1 < z \leq \mu_2 \\ y = 2 & \text{if} & \mu_0 < z \leq \mu_1 \\ y = 1 & \text{if} & z \leq \mu_0 \end{array}$$

where: μ_i are referred to as estimable parameters (thresholds) corresponding to the ordering of injury response from no injury ($y = 1$) to fatality ($y = 5$).

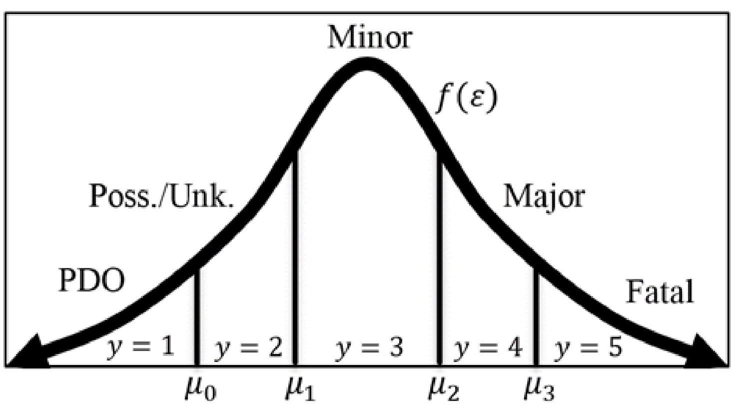


Figure 25: An illustration of ordered probability regression with μ_i as parameter thresholds
after (Washington, Karlaftis and Mannering 2011)

5.3 Injury Severity Results

Before the regression analysis, thematic analysis was performed to investigate patterns and clusters of person-, vehicle-, and crash-level variables used in the injury severity analysis. This analysis was also found useful to get a general understanding of how variables would/should perform. Furthermore, this also helped reduce and remove possibly inconsistent or redundant variables from consideration. Characteristics were classified into relatively homogeneous clusters and later modeled using regression analysis. The groupings of variable clusters for the injury severity analysis are shown in Table 21 of appendix B.

The occupant injury regression focused on characteristics of person-, vehicle-, and crash-circumstantial attributes influencing the injury status of occupants involved in crashes with bridges on county roadways. The regression analysis was also conducted using NLOGIT 5 statistical software. Table 18 shows the ordered probabilistic regression results of occupant injury status for crashes involving bridges on Iowa's county road network.

The results suggested five person-level, three vehicle-level, and three crash-level attributes that are statistically correlated with the ordinal ranking of injuries sustained by occupants of vehicle involved in collisions with bridges on county roadways. In order of appearance, those included the age of occupant, seat protection usage, front seat occupancy, occupant ejection, airbag deployment, vehicle truck configuration, number of vehicle occupants, initial frontal vehicle impact, wet, icy, snowy, or slushy surface condition, drug or alcohol-related incident, and work zone-related incident.

Similar to the crash frequency results, majority of the explanatory variables included in the regression model were indicator (dummy) variables as previously described. Those variables are also highlighted in yellow. Correspondingly, the associated test-statistic (Student's t-statistic) acknowledges that some attributes has significantly stronger explanatory properties than others, as indicated by the quantity of asterisks. The significance of the parameter thresholds implies that the ordinal scaling of the vehicle occupancy injury status confirms to be statistically different from one another. Based on the model results, major findings in regard to the probability of vehicle occupancy injury status are as follows.

Table 18: The random effects ordered probability model results of occupant injury severity (Dependent variable responses are intergers between 1 [No injury] and 5 [Fatal injury])

Explanatory Variable	Variable Classification	Coefficient	test-statistic	Significance
OCCU_AGE	Person Level	0.00792	2.51	**
PROTECT	Person Level	-0.94764	-4.79	***
FRNTSEAT	Person Level	-0.56460	-1.70	*
EJECT	Person Level	0.92275	1.74	*
AIRBAG	Person Level	0.46089	4.32	***
TRUCK	Vehicle Level	-1.06028	-1.77	*
OCCUPANT	Vehicle Level	0.09041	1.39	
IMPFRONT	Vehicle Level	-0.50261	-3.38	***
WISS	Crash Level	-0.30982	-2.59	***
DRUG_ALC	Crash Level	0.50160	3.02	***
WZRELATE	Crash Level	-0.81753	-1.28	
Constant: $-(\mu_0)$		1.34286	3.26	***
Mu 01: (μ_1)		0.66075	9.68	***
Mu 02: (μ_2)		1.66239	11.81	***
Mu 03: (μ_3)		2.39213	11.43	***
Number of Observations		819		
Log-likelihood at zero		-1279.52948		
Log-likelihood at convergence		-623.14423		
Goodness-of-Fit (ρ^2)		0.51299		

Note: ***, **, * ==> Significance at 1%, 5%, 10% level.

1) It was observed that the age of an occupant played a significant role on level of injury sustained. The marginal effects (Table 22 of Appendix B) show that a unit increase in the age of a vehicle occupant reduces the probability of no injury by 0.00316. This implies that the probability of other injury categories (possible/unknown, minor, major, and fatal injury) all increase with age. However, the net effect of increase in age appears ambiguous for major and fatal injuries because of their reduced effects.

2) It was observed that the indicator variable for occupant protection (i.e. seat belt use) played a significant role on the level of injury sustained. The marginal effects show that seat belt use for instance increases the probability of no injury by 0.33671.
3) The indicator variable for seating position (i.e. front seat occupant) was observed to have an overall significant effect on level of injury sustained. The marginal effects show that front seat usage increases the probability of no injury by 0.21386. However, the net effect of front seat occupancy appears ambiguous for major and fatal injuries.
4) The indicator variable for occupant ejection was observed to have an overall significant effect on level of injury sustained. Its marginal effect shows reduction in probability of no injury by 0.32303. However, the net effect of ejection appears ambiguous for major and fatal injuries.
5) The indicator variable for airbag deployment was observed to have significant effect on level of injury sustained. Its marginal effect shows reduction in probability of no injury by 0.18196. Nonetheless, this seems prevailing for nonincapacitating injuries than otherwise.
6) The indicator variable for vehicular configuration was observed to have an overall significant effect on level of injury sustained. The marginal effects show that truck vehicle configuration (as supposed to passenger car or motorcycle) increases the probability of no injury by 0.35887. However, it should be noted that the data represented little-to-no observations of motorcycle crashes.
7) It was observed that the total number of vehicle occupants had no significant effect on level of injury sustained. Nonetheless, a one unit increase in occupancy reduces the probability of a no injury by 0.03607.

8) The indicator variable for vehicle location of impact was observed to have significant effect on level of injury sustained. The marginal effects show that frontal impact increases the probability of no injury by 0.19528. However, the net effect of frontal impact appears ambiguous for major and fatal injuries.

9) The indicator variable for surface condition was observed to have significant effect on level of injury sustained. The marginal effects show that wet, icy, snowy, or slushy surface conditions increases the probability of no injury by 0.12304.

10) The indicator variable for driver condition was observed to have significant effect on level of injury sustained. The marginal effects show that driving under the influence of drug or alcohol reduces the probability of no injury by 0.19413.

11) Lastly, the indicator variable for (work zone) environment was observed to have no significant effect on level of injury sustained. Nonetheless, the marginal effects show that work zone increases the probability of no injury by 0.29480. Additionally, it should be noted that the data represented little-to-no observations of work zone related events.

Overall, the goodness-of-fit statistic ρ^2 of 0.513 for the vehicle occupant injury severity model suggests the appropriate overall fit of the severity dataset results given its distribution. Specifically, the occupant injury severity results suggest evidence in support of an ordinal probabilistic regression. Other variations of ordinal regression, including ordered-logistic or nested-logit models could show comparable results, but those weren't included in this analysis.

In addition to the marginal effects for the injury severity model results (Table 22 of appendix B), Table 23 shows a summary of the parameters elasticities which measure the effect of a 1% change in parameter (X) on the dependent variable (injury severity).

It should be advised that given the unbalanced sample distribution of observations, there are some limitations to the severity model results. Those include:

- Results of the occupant severity model are independent of bridge characteristics.
- Unbalanced sampling of rear seat occupancy; less than 4% of vehicle occupants were rear-seat users.
- Not all vehicles have or require rear/side safety devices.
- By state law, rear-seat occupants are not required to use/wear protective devices such as seat belts.
- Not all age groups are represented; specifically, lower age groups are naturally limited to rear-seat occupancy which may influence severity outcome.
- Airbag deployment is suggestive of more severe crashes at higher speeds.
- 2 observations were characterized as motorcycle occupants (less than half a percent).
- 7 observations were characterized as work zone-related crashes (less than 1%).
- Occupant ejection was not applicable to motorcyclists.
- Total ejection and protective device usage may be complementary although partial ejection may not.

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

This chapter summarizes the major findings and contributions to the state-of-arts, limitations, and recommendations for future research. Section 5.1 of this paper summarizes the major findings and contributions to the state-of-arts in relation to the analysis results. Section 5.2 summarizes the limitations of the results. Section 5.3 summarizes the recommendations for future research.

6.1 Summary

An underlying premise is that vehicular crashes involving bridges are very rare events. In the 10-year analysis period from 2004 to 2013, there were merely 862 single-vehicle bridge crashes in Iowa. Regardless of their generally low occurrences, vehicular crashes involving bridges are more frequent on some bridges more than others. Thus, the primary objective of this research was to explore significant relationships between roadway, bridge, and crash characteristics influencing bridge safety. In conjunction with previous studies, (Chengye and Ranjitkar 2013), (Bigelow, Hans and Phares 2010), (Ardekani, Hauer and Jamei 1992), the crash frequency results confirm that bridge and roadway characteristics (i.e. length, width) in addition to traffic exposure (i.e. AADT) all tend to influence the expected frequency of bridge crashes.

Specifically, bridge crashes are expected to increase as traffic increase (Chengye and Ranjitkar 2013); nonetheless, increase in truck traffic as a percentage of AADT decreases expected crashes as in (Lee and Mannering 1999). In contrast, bridges that are relatively wider than its travel way (positive relative bridge width), and bridges that are on paved roadways are less susceptible to vehicular crashes, as also recommended in (Turner 1984), and (Bigelow, Hans and Phares 2010). Precisely, bridge relative width accounts for all traversable area along the structure, with respect to its approach; thus including lanes and shoulders effects. Though, it is worth mentioning that it is much easier to detect lane/shoulder markings on paved roads than unpaved roads which may influence how vehicles navigate through bridges.

Furthermore, the crash frequency results concur that bridge crashes are observed more frequently with bridges that are older, as in (Mehta, et al. 2015) and (Bigelow, Hans and Phares 2010). Particularly, in combination with other condition characteristics, (superstructure, railing, and alignment conditions) bridges designed to or meeting substandard conditions are more susceptible to crashes over time. Exclusively, from the perspective of design and construction, steel-type bridges show to be a viable alternative for low-volume roads as in (Klaiber, Wipf and Nauman, et al. 2000); however, from the perspective of safety, the crash frequency results show that bridges of steel or timber types tend to be more vulnerable to crashes than concrete bridges.

Several studies (in particular to bridge safety) have considered associative properties of vehicle and crash characteristics on the severity of crashes involving bridges, including (Holdridge, Shankar and Ulfarsson 2005), (Lee and Mannering 1999), (Hall 1982) etc. Nevertheless, none of these go beyond driver or most severe occupant characteristics. That said, the occupant injury severity model considered characteristics of crash, vehicle, and all vehicle occupants. In all, the occupant injury severity results suggested that collisions involving bridges tend to be more severe for unprotected vehicle occupants. Also, concurrent with (Holdridge, Shankar and Ulfarsson 2005), driving under the influence of drug or alcohol increased the probability of more severe injuries. Unexpectedly, the effects of front seat occupancy and airbags deployment seem unconventional given that front seat occupants are the most exposed and airbags are to protect occupants. Specifically, this may be due to the limited sampling of rear seat passengers, and that protective device usage (i.e. seatbelts) for rear passengers are not required by law in Iowa according to the Governors Highway Safety Association (GHSA 2016).

Moreover, though other studies (Hall 1982) suggests that guardrail collisions are frequent at snow-covered surfaces, from the perspective of occupant injuries, bridge collisions during wet, icy, snowy, or slushy road surface conditions however tend to be less severe. Other attributes that tend to decrease the probability of severe injuries include collisions involving frontal impact of the vehicle as supposed to rear or side, and collisions involving trucks. However, more observations are needed to affirm association.

6.2 Limitations

Some important assumptions were made during the data collection and integration procedures. Upon availability, historical data can be of assistance in identifying and evaluating patterns and themes. Thus, a history of several years is needed in order to identify significant patterns, particularly for low volume roads. Nonetheless, such data may become mainly a problem when dealing with inconsistencies and inaccuracies.

Firstly, within the databases may exist inconsistent and inaccurate coding of variables and attributes pertaining to bridge, roadway, and/or crash record information. Nonetheless, the analysis results relied on information reported and recorded in the databases as much as possible. All cases of missing or incomplete attributes were treated as is, and all inconsistent patterns were excluded upon revision.

Secondly, within the crash record, all cases were reliant on the evidence of the reporting officer or individual. Specifically, all information about events leading up to a crash event may rely solely on the reminiscence and character of the reporting individual. As shown in Figure 23, not all persons involved/present at the event of a crash may necessarily be reported within the crash database. This presents a concern when evaluating the likely injury status of all vehicle occupants. Thus, in such cases, missing (not reported) individuals were ignored.

Lastly, there is a certain amount of randomness with roadside crashes. Therefore, a crash analysis should look for patterns of crashes at several sites that share common characteristics, such as roadway features and hazard types, and verify via site inspections. Care must be taken to avoid overreacting to one severe crash at a specific site when there is no established pattern. Otherwise, an expensive corrective action may be constructed to correct a problem that may never recur.

6.3 Future Studies

While the low quantities of bridge crashes on county roads may be indicative of bridge rails and guardrails serving their purpose, the findings of this study demonstrate the need to continue to observe and improve safety performance of bridges on county roadways. Results of this research can be useful to local public agencies regarding guidance for bridge and barrier rail upgrade standards. That said, prescriptive guidelines for bridge rail use on county roads may not be necessary, given the rareness and randomness of crash events.

As suggested by previous literatures ((Dare 1992), (Gates and Noyce 2005), (Seitz and Salfrank 2014)), considering categorical thresholds may be useful for establishing practical requirements of when and how county engineers should upgrade bridges. Also, additional efforts toward future investigations could include modeling injury severity as nested (logit) regression to minimize ambiguities within model results.

In closing, special consideration/emphasis may be placed on bridges possessing certain characteristics because they expect higher crashes, although a relatively small proportion of bridges may actually possess these characteristics. Nonetheless, these processes require the development of separate allocation-of-fund that are in dire need of by bridge engineers.

APPENDIX A

CHAPTER 2 APPENDICES

Table 19: FHWA Bridge rail memorandum

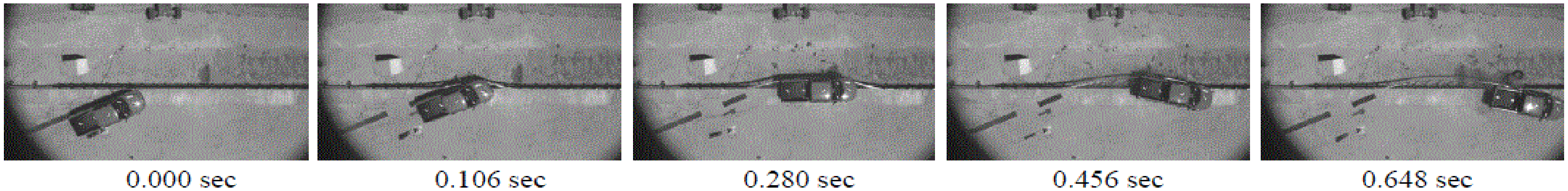
FIGURE NUMBER	BRIDGE RAILING	RAILING HEIGHT (IN.)	TEST VEHICLE	IMPACT SPEED (MPH)	IMPACT ANGLE DEGREES	Meets NCHRP 230	PERFORMANCE LEVEL	NCHRP 350 EQUIVALENT TEST LEVEL FROM FHWA MEMO 3	REFERENCES	FHWA Bridge Rail Memo #1	FHWA Bridge Rail Memo #2	FHWA Bridge Rail Memo #3
W-BEAM BRIDGE RAIL												
1	Texas Type T6 (Tubular W-beam)	27	2,280 lb. Car 4,500 lb. Car 2,000P Pickup	58 61.6 100 kph	14 27.5 25	Yes		TL-2	1,2 1,2 3	3		1-3
2	West Virginia W-beam Retrofit Railing for Concrete Baluster designs, curb mounted	28.5	817 kg Car 2452 kg Pickup	80 kph 72 kph	20 20		Originally tested to PL-1	TL-2	4			3-11
THRIE-BEAM BRIDGE RAIL												
3	NCHRP SL1 Thrie Beam, Wood Posts	32	2,250 lb. Car 2,250 lb. Car 4,500 lb. Car	63 60.1 61.9	18.7 15.9 14.5	Yes		TL-2	2,5	1		1-1
4	NCHRP SL1 Thrie Beam, Steel Posts	32	1,987 lb. Car 2,250 lb. Car 2,250 lb. Car 20,000 lb. Bus	61.4 58.6 60 44.7	14.1 16 16 7.7	Yes		TL-2	2,5	2		1-2
5	Nebraska Tubular Thrie Beam	32	1,970 lb. Car 4,700 lb. Car	61.4 58.4	20 24.3	Yes		TL-3	6,7	11		1-12
6	Oregon Side-Mounted Thrie Beam	27	1,970 lb Car 5,737 lb Pickup	52.2 46.1	19.7 20.8	Yes	PL-1	TL-2	8,9,10		1-1	2-1
7	Washington 10 gage Thrie Beam Retrofit for Balusters Curb/Sidewalk	30	1,840 lb Car 4,725 lb Car 5,400 lb Pickup	58.8 60.7 66.3	19.5 15.6 19.4	Yes	PL-1	TL-2	11		1-8	2-7
8	California Thrie Beam	32	1,936 lb Car 5,565 lb Pickup	48.7 44.9	18.3 21	Yes	PL-1	TL-2	12,13		1-9	2-8
9	Missouri Thrie Beam and Channel (top mounted)	30-5/8", 30.625"	1,984 lb Car 4,495 lb Car	59.6 60.9	15 24	Yes		TL-3	14,15		1-10	2-13
10	Michigan 10 gage retrofit on Curb/Sidewalk (Michigan R4 Retrofit Bridge Rail)	34	1,972 lb Car 5,724 lb Pickup 18,000 lb Truck	61.5 60.6 49	20 20 15	Yes	PL-2	TL-4	16		1-15	2-15
11	Delaware Thrie-beam Retrofit Railing (curb-mounted)	32	8,000 kg Single-Unit Truck	80 kph	15		Tested to NCHRP 350	TL-4	4			3-24
METAL TUBE BRIDGE RAIL												
Aluminum Tube Bridge Rail												
12	Aluminum Tru-Beam (Modified AASHTO BR5)	32	2,150 lb. Car 4,500 lb. Car	61.3 58.9	21.5 27.2	Yes		TL-2	17,18	4		1-4
13	Foothills Parkway Aluminum Bridge Rail	33	2,069 lb Car 5,465 lb Pickup 5,565 lb Pickup	52 46.6 45.7	22 20.7 22.7		passed PL-1, NCHRP 230 failed, anchorage not long enough passed, corrected ancchorage, PL-1	TL-2	19,20		2-22	3-12
Steel Tube Bridge Rail, Attached to Bottom of Deck												
14	Texas Energy Absorbing Bridge Rail	27	1,972 lb. Car 4,500 lb. Car	62.6 61	16 25	Yes		**NCHRP 230**	21,22,23	6		
Steel Tube Bridge Rail, Attached to Side of Deck												
15	California Type 18 (See-Through, Collapsing Ring)	36	1,850 lb. Car 4,530 lb. Car	59.7 60.7	12 23	Yes		**NCHRP 230**	24,13	17		
16	Collapsing Ring Bridge Railing	59	2,090 lb. Car 4,400 lb. Car 40,000 lb. Bus 40,000 lb. Tractor-Trailer 70,000 lb. Tractor-Trailer	55.7 62 53.9 57 44.4	23.5 22.7 15.1 15.6 10	Yes		**NCHRP 230**	25,26,27,28	21		
17	Ohio Box Beam Rail (W-beam backed up with box beam)	27	1,980 lb. Car 4,790 lb. Car	60.6 60	19.6 25	Yes		TL-2	6	8		1-6
18	California Type 115	30	1,965 lb Car 5,635 lb Pickup	59 64.2	19 21	Yes	PL-1	TL-2	29,13		1-7	2-6
19	Illinois Side Mounted Bridge Rail	32	1,970 lb Car 5,565 lb Pickup 18,000 lb Truck	59.9 60.4 51.4	20.1 20.4 14.7		tested to PL-2	TL-4	8,30		2-6	3-22
Steel Tube Bridge Rail, Attached to Top of Deck												
20	Texas T101 Bridge Rail	27	2780 lb. Car, 4,660 lb. Car 4,630 lb. Car 6,900 lb. Bus 19,940 lb. Bus 20,010 lb. Bus 31,880 lb. Bus	57.3 60.2 59.8 53.4 55.3 52 58.4	15 15 25.8 15 15.2 13.2 16	Yes		TL-3	18,31,32,33,2	7		1-11
21	Texas 421 Aesthetic Steel Pipe Bridge Rail	32	1,800 lb Car 4,500 lb Car	59.7 62.4	21.4 26.6	Yes		TL-2	34,35		1-12	2-11
22	Washington, D.C. Historic Bridgerail [curb-mounted retrofit]	27	1,965 lb Car 5,565 lb Pickup	49.7 47.7	21.5 20.6		PL-1	TL-2	36		2-23	3-15
Steel Tube Bridge Rail, Attached to Parapet												
23	AASHTO BR2 (California Type 9)	27	1,929 lb. Car 4,540 lb. Car	60.9 57	13.1 26	Yes		TL-2	2, unknown 37	5		1-5
24	North Carolina - Standard 1 Bar Metal Rail	32	1,990 lb. Car 4,660 lb. Car 19,920 lb. Bus	59.7 60 57.3	18.8 25 14.8	Yes		TL-2	6,8 6,8 18,31,32,33	13		1-10
25	Modified Texas C202 Bridge Rail	54	1,918 lb Car 4,400 lb Car 79,770 lb Van Type Tractor Trailer	61.3 59.4 49.1	21 25.9 15	Yes	**Special PL-4** (originally PL-3)	TL-6	38 38 39		1-25	2-23
26	BR27D-two steel Rails on 18" concrete parapet w/ 8-in curb and 5-ft sidewalk	42	1,967 lb Car 5,565 lb Pickup	51.7 45.3	20.8 20.2		PL-1	TL-2	8,40,41		2-2	3-16
27	BR27D-flush-mounted	42	1,970 lb Car 5,566 lb Pickup	51.2 45.6	20.5 18.8		PL-1	TL-2	8,40,41		2-3	3-17
28	BR27C-single steel rail on 24" concrete parapet w/ 8-in. curb and 5-ft sidewalk	42	1,965 lb Car 5,568 lb Pickup 18,000 lb Truck	61.7 62.6 51	18.7 19.4 13.7		PL-2	TL-4	8,40,42		2-4	3-18
29	BR27C-flush-mounted	42	1,970 lb Car 5,570 lb Pickup 18,000 lb Truck	60.3 55.3 52.5	19.8 19.6 12.8		PL-2	TL-4	8,40,42		2-5	3-19
30	Minnesota Combination Bridge Rail, Design #3	36"	18,000 lb Truck 4,420 lb Pickup 4,442 lb Pickup 1,960 lb Car	50.8 60.6 62.5 61	16.2 25.5 25.9 20.6		Tested to NCHRP 350	TL-4	43			3-26
Steel Tube Bridge Rail, Attached to Curb												
31	Oregon - 2 Tube Mounted Rail (Curb Mounted)	32	1,994 lb. Car 4,640 lb. Car	58.6 60	18.8 25	Yes		TL-2	6,7	12		1-9
32	Wyoming 2-Tube, Curb Mounted (6")	29 in., 740 mm	1,968 lb Car 4,510 lb Car 2000 kg Pickup	61.1 63.3 101.7 kph	20 25 25.2	Yes		TL-3	44,45 44,45 46		1-4	2-14
33	Wyoming 2-tube steel railing on 150 mm Curb (32.7" railing, on 5.9" curb)	830 mm (32.7")	896 kg Car 2000 kg Pickup 8000 kg Truck	97.8 kph 101.0 kph 79.7 kph	20.8 24.9 15.9		Tested to NCHRP 350	TL-4	47		2-25	3-25
34	Illinois 2399, 2-Rail on Curb	32	1,961 lb Car 5,797 lb Pickup 18,000 lb Truck	58.7 63.6 50.8	20 19.2 15.1	Yes	PL-2	TL-4	8,48,15 8,48,15,49 8,48,15,49		1-21	2-19
35	GW (George Washington) Memorial Parkway Steel Tri-Rail on curb	42	2,010 lb Car 1,936 lb Car 5,400 lb Pickup	54.4 52.6 46.6	21 22.6 22.7	Yes	failed PL-1 passed PL-1, NCHRP 230 passed PL-1, NCHRP 230	TL-2	50,51		2-21	3-13
36	New England Transportation Consortium (NETC) 2-rail curb-mounted railing, steel	864 mm (34 in)	894 kg (1970 lb) Car 2258 kg (5568 lb) Pickup 8000 kg (17,621 lb) Pickup	100.9 kph (62.7 mph) 92.2 kph (57.3 mph) 81.7 kph (50.8 mph)	20.6 20.6 15.5		tested to PL-2 and TL-4	TL-4	36		2-18&19	3-23

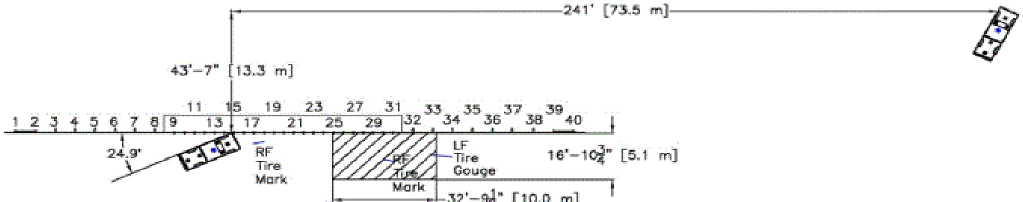
Source: http://safety.fhwa.dot.gov/roadway_dept

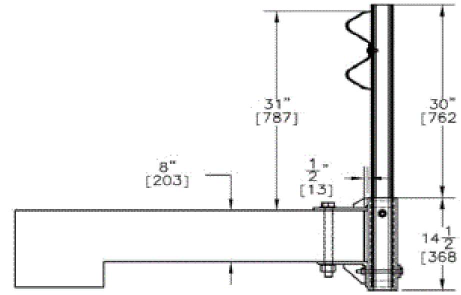
Table 19: FHWA Bridge rail memorandum (Continued)

FIGURE NUMBER	BRIDGE RAILING	RAILING HEIGHT (IN.)	TEST VEHICLE	IMPACT SPEED (MPH)	IMPACT ANGLE DEGREES	Meets NCHRP 230	PERFORMANCE LEVEL	NCHRP 350 EQUIVALENT TEST LEVEL FROM FHWA MEMO 3	REFERENCES	FHWA Bridge Rail Memo #1	FHWA Bridge Rail Memo #2	FHWA Bridge Rail Memo #3
VERTICAL CONCRETE PARAPET (Open or Closed) / General												
37	Modified Kansas Corral (Open Concrete Beam & Post)	27	1950 lb Car 4,690 lb. Car	59 59.2	18.9 24.9	Yes		TL-2	6,7,51,52	9		1-7
38	Oklahoma Modified TR-1 Bridge Rail (Open Concrete Beam & Post)	29	1,980 lb. Car 4,660 lb. Car	58.7 59.1	18.9 25.4	Yes		TL-2	6,7	10		1-8
39	Texas T202 Concrete Beam and Post	27	1,800 lb Car 4,500 lb Car	59.4 59.2	15 26	Yes		TL-2	53,2		1-2	2-2
40	Federal Lands Modified Kansas Corral	27	1,990 lb Car 5,410 lb Pickup	51 46.6	20.5 20	Yes	PL-1	TL-2	51,52,6		1-3	2-3
41	Nebraska Concrete Beam and Post	29	1,971 lb Car 4,669 lb Car	59.8 57.6	21 26	Yes		TL-2	54,55,56		1-5	2-4
42	Iowa Concrete Beam and Post	29	1,922 lb Car 4,662 lb Car	60.1 58.5	20.5 25	Yes	PL-1	TL-2	54		1-6	2-5
43	Texas T411 Aesthetic Concret Baluster	32	1,780 lb Car 4,500 lb Car 2000P lb Pickup	60.2 62.2 100 kph	21.2 26 25	Yes		TL-2	57,58 61 59		1-11	2-10
44	Aesthetic Stone Masonry - Faced Concrete Bridge Rail (BW [Baltimore-Washington] Parkway, Smooth Stone)	32	4,694 lb Car	60.4	25	Yes	This is the same system in part II and part III of the 3rd FHWA memo.	TL-2 TL-3	60		1-13	2-12 / 3-10
45	Iowa Concrete Block Retrofit	32	2,014 lb Car 2,014 lb Car 5,551 lb Pickup 17,814 lb Truck 17,814 lb Truck	56.8 62.5 62.3 44.8 49.9	20 20 20 15.6 15.1	Yes	PL-2 Rolled Over Rolled Over	TL-4	61,15		1-16	2-16
46	32-in Vertical Concrete Parapet	32	1,965 lb Car 5,759 lb Pickup 18,000 lb Truck	60.5 59.7 50	21 20.2 14	Yes	PL-2	TL-4	8,62,48,15 8,63,48,15 8,48,15		1-17	2-17
47	42-in Vertical Concrete Parapet	42	50,050 lb Tractor/Trailer 36,000V kg Truck	51.4 80.1 kph	16.2 14.5	Yes	Rolled Over PL-3	TL-6	8,64,15 8,65,15		1-22	2-20
48	Texas C411 42" Concrete Aesthetic Baluster on 8-in high curb and 6-ft wide sidewalk	42	1,970 lb Car 4,500 lb Car	45.5 47	20.1 25.4	Yes		TL-2	58,66		2-16	3-9
49	Natchez Trace Concrete Bridgerail [post and beam]	32.5	2,015 lb Car 5,665 lb Pickup	51.5 45.2	19.5 22.4		passed PL-1, NCHRP 230 passed PL-1, NCHRP 230	TL-2	50,51		2-20	3-14
50	Nebraska Open Concrete Bridgerail (modified from earlier TL-2 design with expansion gap)	29	5,300 lb Pickup 5,390 lb Pickup 8165 kg (18,000 lb) Truck 8165 kg (18,000 lb) Truck 2447 kg (5,394 lb) Pickup 2449 kg (5,399 lb) Pickup	47.7 45.9 78.1 kph (48.5 mph) 83.5 kph (51.9 mph) 96.2 kph (59.8 mph) 98.2 kph (61.0 mph)	20 20 17.1 16.8 21.7 20		PL-2	TL-4	55 55 56 56 56 56		2-15	3-20
51	32-in High Kansas Corral	32	18,040 lb Truck	51.5	15			TL-4	52		2-14	new
New Jersey Barrier												
52	California Type 25 (N.J. Concrete Safety Shape)	32	4,540 lb. Car 4,540 lb. Car 4,540 lb. Car	38 65 63	7 7 25	Yes		NCHRP 230	67,68,69 67,68,69,13 67,68,69,70	14		
53	N.J. Concrete Safety Shape	32	1,970 lb. Car 1,968 lb. Car 4,500 lb. Car 18,240 lb. Truck 19,990 lb. Bus 20,000 lb. Bus 20,270 lb. Bus 40,000 lb. Bus 40,000 lb. Bus 40,000 lb. Bus 40,020 lb. Bus 40,030 lb. Bus 40,030 lb. Tractor-Trailer	60.4 61.3 60.1 60.1 60.9 57.7 61.6 41.6 51.6 52.9 54 54 53	15 20 25.2 15 16 15 15 11.5 6.6 16 16.2 14 15	Yes		TL-4	71 71,72 67,73 unknown 71 unknown 71 67,74,73 67,74,73 67,74,73 71 71 71	15		1-15
54	32-in NJ-Shape	32	5,724 lb Pickup 18,000 lb Truck	57.7 51.6	20.6 15.5		PL-2 Rolled Over	TL-4	8,75,48,15 8,76,48,15		1-18	
55	L.B. Foster Precast NJ-Shape, bolted down with Kelken-Gold Polyester Resin Bolts	34	18,000 lb Truck	51.7	15	Yes	Rolled Over PL-2	TL-4	77		1-20	2-18
56	Missouri 30" NJ Concrete Barrier (to test effect of 2" asphalt overlay on standard barrier height)	30	18,011 lb Truck 1,759 lb Car 5,460 lb Pickup	52.5 62.5 63.5	16.1 20 20		PL-2	TL-4	78,72		2-17	3-21
New Jersey Barrier w/ RAIL												
57	California Type 20 (N.J. Safety Shape with Rail)	39	4,980 lb. Car 4,980 lb. Car 4,900 lb. Car 4,980 lb. Car 4,900 lb. Car	45 66 64 64 66	7 7 15 7 25	Yes		TL-3	79,80,81	18		1-13
58	Nevada Safety Shape Parapet	39	1,911 lb. Car 4,650 lb. Car 40,000 lb. Bus	60.7 61.4 58.9	19.3 24.9 16.4	Yes		TL-3	6	19		1-14
59	New Jersey Turnpike Heavy Vehicle Barrier (Extended N.J. Safety Shape)	42	2,118 lb. Car 4,880 lb. Car 80,180 lb. Tractor-Trailer	59.9 58.6 52.1	14.5 16.5 16.5	Yes		TL-6	82,83,84,5	20		1-17
60	Texas T5 Modified (Extended N.J. Safety Shape)	approx 90" above deck	80,120 lb Tank Type Tractor-Trailer	51.4	15	Yes		TL-6	85	22		1-18
61	Texas Type HT (Modified T5)	50	80,080 lb Truck	48.4	14.5	Yes	Rolled Over Special PL-4	TL-6	86		1-24	2-22
F-SHAPE CONCRETE BARRIER / SINGLE SLOPE												
62	F Profile Concrete Safety Shape	32	2,250 lb. Car 4,370 lb. Car 4,500 lb. Car	56.4 61.4 62.9	14.3 15.2 25	Yes		TL-4	67,73	16		1-16
63	32-in F-Shape	32	1,966 lb Car 5,780 lb Pickup 18,050 lb Truck speed low 18,050 lb Truck speed low 18,000 lb Truck	60.1 65.4 46.7 47.3 52.1	21.4 20.4 15 15.3 14.8	Yes	PL-2	TL-4	8,87,48,15 8,88,49,15 8,89,15 8,90,15 8,91,15		1-19	
64	42-in F-Shape	42	40,560 lb Bus 50,000 lb Tractor/Trailer	55.7 52.2	15.7 14	Yes	PL-3	TL-6	8,92,64 8,93,64		1-23	2-21
65	Single Slope Concrete Bridge Rail	32	2076 kg (4,573 lb) Pickup 8172 kg (18,000 lb) Single-Unit Truck 8172 kg (18,000 lb) Single-Unit Truck	97.2 kph (60.4 mph) 82.1 kph (51.0 mph) 82.5 kph (51.3 mph)	25.5 10 17.9		Tested to NCHRP 350	TL-4	94,36,95,96 94,36,96 94,36,96			3-27
TIMBER BRIDGE RAIL												
66	Glu-Lam Wood on Wood Deck	35	1,983 lb Car 5,419 lb Pickup	59.2 47.5	20 20	Yes	PL-1	TL-2	20,97		1-14	2-9
67	Timber Rail #1 on Transversely Laminated Timber Deck	27	1,965 lb Car 5,665 lb Pickup	50.1 46	18.6 20.3		PL-1	TL-2	98,99		2-9	3-1
68	Timber Rail #2 on Transversely Laminated Timber Deck	27	1,967 lb Car 5,570 lb Pickup	49.7 46.1	21 19.1		PL-1	TL-2	100,99		2-10	3-2
69	Timber Rail #3 on Longitudinally Laminated Timber Deck	27	1,968 lb Car 5,567 lb Pickup	51 47	20.4 20.5		PL-1	TL-2	101,99		2-12	3-3
70	Steel System-Thrie Beam on steel posts	81.28 cm	2,542 kg Pickup	71.2 kph	19.1		MWRSF PL-1	TL-2	102,103		2-11	3-4
71	Curb System - Glu-Lam timber rail w/curb	81.28 cm	2,452 kg Pickup	71 kph	23.4		MWRSF PL-1	TL-2	102,103		2-7	3-5
72	Shoe Box System-Glu-Lam rail w/out curb	81.28 cm	839 kg Car 2,452 kg Pickup	80.7 kph 72.4 kph	21.5 21.8		MWRSF PL-1	TL-2	102,103		2-8	3-6
73	TBC-8000 - Thrie - beam w/stiffened steel posts	84.5 cm	8,165 kg Truck	76.3 kph	16.1		MWRSF PL-2	TL-4	104,105		2-24	3-7
74	GC-8000 Glu-Lam timber rail w/curb	83.8 cm	2,087 kg Pickup 8,164 kg Truck	98.0 kph 82.4 kph	24.9 16.8		MWRSF PL-2	TL-4	104,105		2-13	3-8

Source: http://safety.fhwa.dot.gov/roadway_dept







- Test Agency .... MwRSF
- Test Number MGSBR-1
- Date 6/18/2009
- MASH Test Designation 3-11
- Test Article .... MGS Bridge Rail with approach MGS Guardrails
- Total Length .... 175 ft (53.3 m)
- Key Component – MGS Bridge Rail
  - Post Type S3x5.7 (S76x8.5)
  - Post Spacing .... 3 ft - 1½ in. (953 mm)
  - Post-to-rail Connection .... 5/16-in. (7.9-mm) dia. ASTM A307A Grade A Bolt
  - Mounting Bracket .... 4x4x⅜ (102x102) ASTM A572 Tube
  - Deck Connection .... 1-in. (25.4-mm) dia. Grade 5 Bolt
- Key Component – Simulated Bridge Deck
  - Thickness (outer edge) .... 8 in. (203 mm)
  - Minimum 28-day Concrete Strength .... 4,000 psi (27.6 MPa)
- Vehicle Model .... 2004 Dodge Ram 1500 Quad Cab Pickup Truck
  - Curb .... 5,134 lb (2,329 kg)
  - Test Inertial .... 5,005 lb (2,270 kg)
  - Gross Static .... 5,174 lb (2,347 kg)
- Impact Conditions
  - Speed .... 61.9 mph (99.6 km/h)
  - Angle .... 24.9 degrees
  - Impact Location .... 15 ft - 9½ in. (4.81 m) US of post no. 20 splice
- Exit Conditions
  - Speed .... 34.5 mph (55.5 km/h)
  - Angle .... 20.4 degrees
  - Exit Box .... Pass
- Vehicle Stability .... Satisfactory
- Vehicle Stopping Distance .... 241 ft (73.5 m) downstream
  43 ft - 7 in. (13.3 m) behind edge of bridge deck
- Vehicle Damage .... Moderate
  - VDS 11-LFQ-3
  - CDC 11-LYEW-4
  - Maximum Interior Deformation .... 1¾ in. (44 mm), left side panel
- Test Article Damage .... Moderate

- Test Article Deflections
  - Permanent Set .... 31⅞ in. (810 mm)
  - Dynamic .... 48.9 in. (1,242 mm)
  - Working Width .... 53.2 in. (1,351 mm)
- Angular Displacements (EDR-4)
  - Roll .... -15.3 degrees
  - Pitch .... -5.6 degrees
  - Yaw .... 37.8 degrees
- Angular Displacements (DTS)
  - Roll .... -14.0 degrees
  - Pitch .... -5.4 degrees
  - Yaw .... 39.8 degrees

Transducer Data					
Evaluation Criteria		Transducer			MASH Limit
		EDR-4	DTS	EDR-3	
OIV ft/s (m/s)	Longitudinal	-16.94 (-5.16)	-16.86 (-5.14)	-18.84 (-5.74)	≤ 40 (12.2)
	Lateral	13.27 (4.04)	14.23 (4.34)	14.18 (4.32)	≤ 40 (12.2)
ORA g's	Longitudinal	-10.61	-10.44	-12.55	≤ 20.49
	Lateral	5.42	6.33	5.61	≤ 20.49
THIV – ft/s (m/s)		20.66 (6.30)	21.03 (6.41)	N/A	not required
PHD – g's		10.64	10.50	N/A	not required
ASI		0.53	0.57	0.64	not required

Figure 26: MSG Bridge rail test results and summary, Truck

Source: (Thiele, et al. 2011)

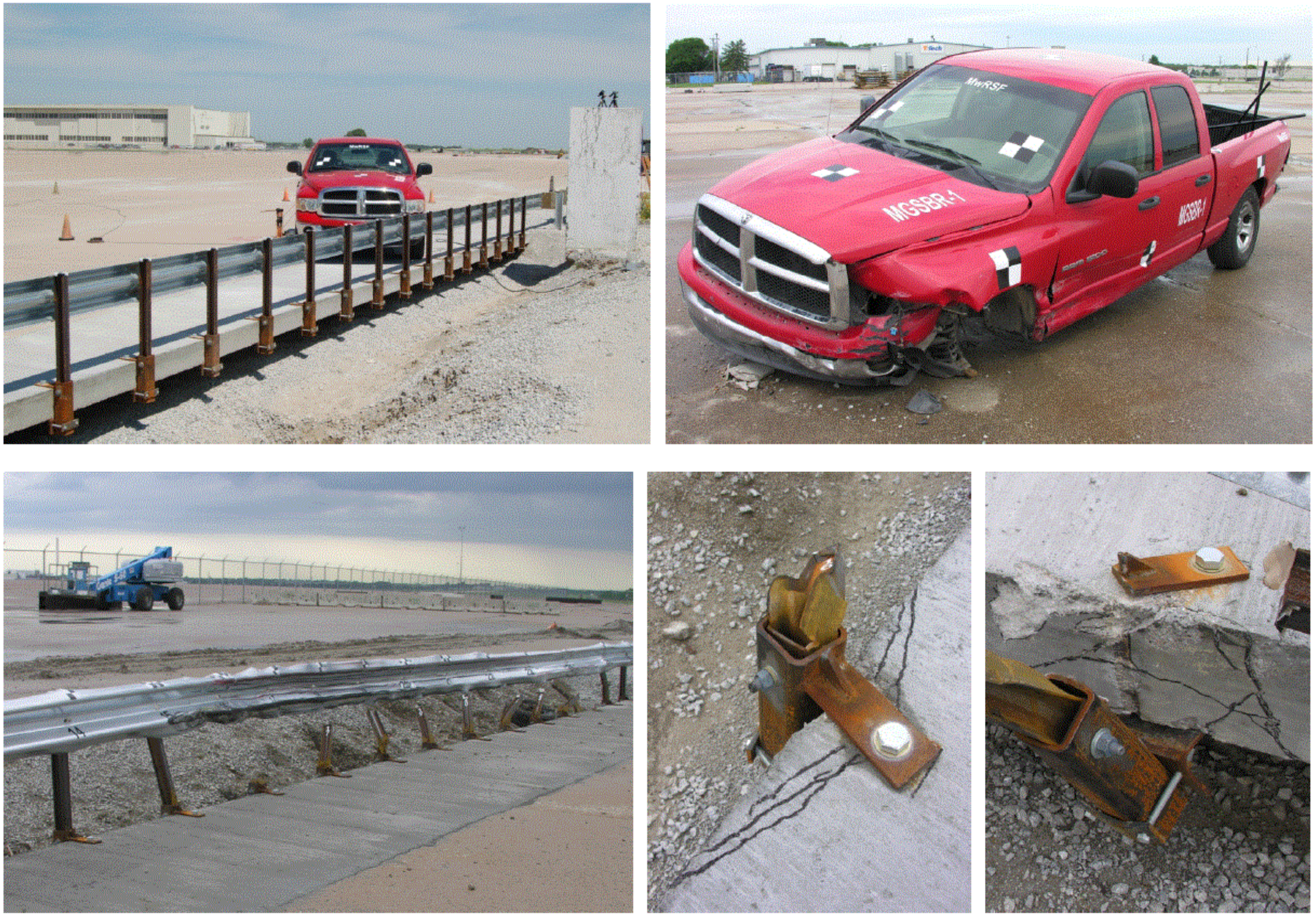
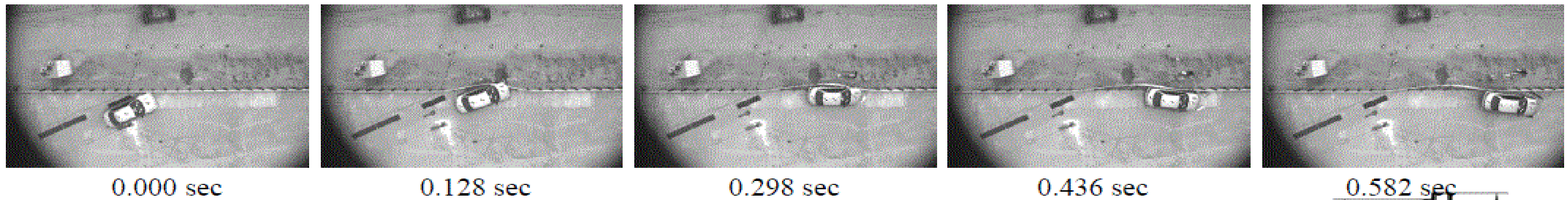
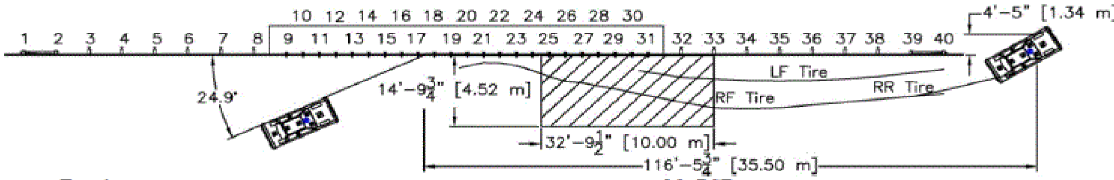
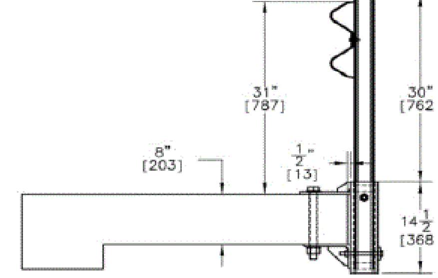


Figure 27: MSG Bridge rail system and vehicle (truck) damages
Source: (Thiele, et al. 2011)







- Test Agency .... MwRSF
- Test Number MGSBR-2
- Date 6/26/2009
- MASH Test Designation 3-10
- Test Article .... MGS Bridge Rail with approach MGS Guardrails
- Total Length .... 175 ft (53.3 m)
- Key Component – MGS Bridge Rail
  - Post Type S3x5.7 (S76x8.5)
  - Post Spacing .... 3 ft - 1½ in. (953 mm)
  - Post-to-rail Connection .... $^{5}/_{16}$-in. (7.9-mm) dia. ASTM A307A Bolt
  - Mounting Bracket .... 4x4x⅜ (102x102x102) ASTM A572 Tube
  - Deck Connection .... 1-in. (25.4-mm) dia. Grade 5 Bolt
- Key Component – Simulated Bridge Deck
  - Thickness (outer edge) .... 8 in. (203 mm)
  - Minimum 28-day Concrete Strength .... 4,000 psi (27.6 MPa)
- Vehicle Model .... 2003 Kia Rio Passenger Car
  - Curb .... 2,408 lb (1,092 kg)
  - Test Inertial .... 2,416 lb (1,096 kg)
  - Gross Static .... 2,585 lb (1,173 kg)
- Impact Conditions
  - Speed .... 62.3 mph (100.2 km/h)
  - Angle .... 24.9 degrees
  - Impact Location .... 7 ft - 9 in. (2.36 m) US of the splice at post no. 20
- Exit Conditions
  - Speed .... 27.7 mph (44.6 km/h)
  - Angle .... 10.9 degrees
  - Exit Box .... Pass
- Vehicle Stability .... Satisfactory
- Vehicle Stopping Distance .... 116 ft - 5¾ in. (35.5 m) downstream
  4 ft - 5 in. (1.3 m) behind front face of guardrail
- Vehicle Damage .... Moderate
  - VDS 11-LFQ-4
  - CDC 11-LYEW-5
  - Maximum Interior Deformation .... 1¼ in. (32 mm), front of floor pan
- Test Article Damage .... Moderate
- Test Article Deflections
  - Permanent Set .... 20 in. (508 mm)
  - Dynamic .... 28.0 in. (712 mm)
  - Working Width .... 33.8 in. (859 mm)
- Angular Displacements (EDR-4)
  - Roll .... -8.7 degrees
  - Pitch .... -8.2 degrees
  - Yaw .... 34.7 degrees
- Angular Displacements (DTS)
  - Roll .... -12.7 degrees
  - Pitch .... -5.0 degrees
  - Yaw .... 35.1 degrees
- Transducer Data

Evaluation Criteria		Transducer			MASH Limit
		EDR-4	DTS	EDR-3	
OIV ft/s (m/s)	Longitudinal	-24.40 (-7.44)	-22.90 (-6.98)	-25.29 (-7.71)	≤ 40 (12.2)
	Lateral	16.54 (5.04)	16.38 (4.99)	17.94 (5.47)	≤ 40 (12.2)
ORA g's	Longitudinal	-7.69	-7.41	-8.65	≤ 20.49
	Lateral	6.58	7.34	7.39	≤ 20.49
THIV – ft/s (m/s)		28.50 (8.69)	28.04 (8.55)	N/A	not required
PHD – g's		9.93	9.90	N/A	not required
ASI		0.79	0.78	0.88	not required

Figure 28: MSG Bridge rail test results and summary, Car

Source: (Thiele, et al. 2011)

Figure 29: MSG Bridge rail system and vehicle (car) damages
Source: (Thiele, et al. 2011)

INSTRUCTIONAL MEMORANDUMS

To Local Public Agencies

To:	Counties and Cities	Date: July 18, 2013
From:	Office of Local Systems	I.M. No. 3.213
Subject:	Traffic Barriers (Guardrail and Bridge Rail)	

Contents: This Instructional Memorandum (I.M.) provides guidelines for determining the need for traffic barriers at roadway bridges and culverts. This I.M. also provides guidelines for upgrading bridge barrier rails. This I.M. includes the following attachments:

Attachment A - Bridge Barrier Rail Rating System (Word)

Other obstructions, within the right-of-way and clear zone, should be reviewed for removal, relocation, or installation of a traffic barrier; or the “do nothing” option based on a cost-effectiveness approach. Refer to I.M. 3.215, Clear Zone Guidelines.

APPROACH GUARDRAIL

In general, approach guardrail should be installed at the following:

1. On newly constructed bridges on the Farm-to-Market system, guardrail should be installed on all 4 corners; except bridges located within an established speed zone of 35 mph or less.

2. On Federal-aid bridges constructed or rehabilitated on rural local roadways, guardrail should be installed on the approach corner in both directions (right side in each direction); except bridges located within an established speed zone of 35 mph or less. Consideration should be given to shielding the trailing corner (left side in each direction) if it is located on the outside edge of a curve. Approach guardrail shall also be upgraded when bridge barrier rail is upgraded.

3. On 3R projects on the Farm-to-Market System, all four corners within the project limits. Existing W-beam installations that are flared and anchored at both ends may be used as constructed without upgrading to current standards.

4. Culverts with spans greater than 6 feet (circular pipe culverts greater than 72 inches in diameter), if it is impractical to extend beyond the clear zone and grates are not utilized.

The FHWA will participate in guardrail, including at all four corners of a bridge, if desired by the county.

Design Exceptions

Design exceptions (refer to I.M. 3.218, Design Exception Process) to not install guardrail at bridges or culverts will be considered if all of the following conditions exist:

1. Current average daily traffic (ADT) at structure is less than 400 vehicles per day.

2. Structure width is 24 feet or greater.

3. Structure is on tangent alignment.

4. Benefit/cost Ratio is less than 0.80.

5. Bridge width is wider than the approach roadway width.

Design exceptions are also possible for guardrail installations that may not be considered crashworthy. For example, standard approach guardrail may not be feasible for a structure located in close proximity to an intersection or entrance, so the guardrail may need to be curved around the radius. Depending on the radius, such an installation might not be considered crashworthy. However, compared to placing a crash cushion or doing nothing, curving the guardrail around the radius may provide the best compromise of cost and safety.

Work with the appropriate Administering Office for more guidance on these issues.

BRIDGE BARRIER RAIL

On newly constructed bridges, the bridge barrier rail shall be constructed to the current acceptable standards (includes SL-1 type rail on structures with less than 1000 vpd).

On Federal-aid bridge rehabilitation projects involving the superstructure, any substandard bridge barrier rail, as well as approach guardrail, shall be upgraded. For Federal-aid bridge rehabilitation projects that do not involve the superstructure, it is strongly recommended that the bridge barrier rail, as well as approach guardrail, be upgraded to the current acceptable standards.

Bridge barrier rail that is coded 0 on Item 36A, Bridge Railings, on the SI&A form of the National Bridge Inspection Standards (NBIS), does not meet current acceptable standards and shall be reviewed for upgrading as part of the 3R projects. Use the “Bridge Barrier Rail Rating System”, see Attachment A to this I.M., to assist in determining if a bridge barrier rail should be upgraded with the 3R project and to what extent it should be upgraded. Any bridge which is programmed in the County Five Year Plan for replacement or rehabilitation may not require upgrading as part of the 3R roadway project.

The Bridge Barrier Rail Rating System assigns points to five factors (Crashes, ADT, Width, Length and Type of bridge rail). The sum of these factors will indicate the degree or amount of upgrading required, if any. The crash factor involves crashes (property damage only, personal injury, and fatality) in the last 5 years. The types of bridge barrier rail are from various county bridge standards. If the existing bridge barrier rail is not an old standard, then determine which type it is similar to and assign the corresponding points.

Consideration should be given to extending the guardrail through the bridge on short bridges or bridges which have no end posts. This may be less costly than attaching the guardrail as per the Iowa DOT Standard Road Plans or constructing an end post.

BRIDGE BARRIER RAIL RATING SYSTEM

Name: ______________________________ Date: ______________________________

Bridge ID: ______________________________ County / City: ______________________________

FHWA No.: ______________________________ ADT: ______________________________

Main Span Materials & Design (Item 43): ______________________________

Location: ______________________________

An upgrade to the bridge barrier rails is not required when the "Total Points" are under 25. The following is a list of the required upgrade to the bridge barrier rails relative to the "Total Points":

25 - 50 Points - delineation according to Iowa DOT Standard Road Plans
51 - 75 Points - block out with Thrie-Beam to curb edge
> 75 Points - retrofit

	POINTS	POINTS GIVEN
1. Crashes (in the past 5 years):		
A. None	0	
B. 1 Property Damage Only (PDO)	5	
C. 1 Personal Injury (PI)	10	
D. 1 Fatality (F), 2 PDO, or 1 PI and 1 PDO	15	
E. $\geq$ 2 F, $\geq$ 2 PI, or $\geq$ 3 PDO	20	______
2. ADT (current year):		
A. <200	0	
B. 200-299	5	
C. 300-399	10	
D. 400-750	15	
E. >750	20	______
3. Bridge width (curb-to-curb) (feet):		
A. $\geq$ 30	0	
B. 28	5	
C. 24	10	
D. 22	15	
E. $\leq$ 20	20	______
4. Bridge Length (feet):		
A. <50	0	
B. 50-99	5	
C. 100-149	10	
D. 150-200	15	
E. > 200	20	______
5. Type:		
A. Aluminum Rail (1967 Standard)	0	
B. Steel Box Rail (1964 Standard)	5	
C. Formed Steel Beam Rail (1951 or 1957 Standards)	10	
D. Steel Rail (1941 Standard) or Concrete Rail (1928 Standard)	15	
E. Angle Handrail (1928 Standard)	20	______
Total Points =		______

APPENDIX B

CHAPTERS 4 AND 5 APPENDICES

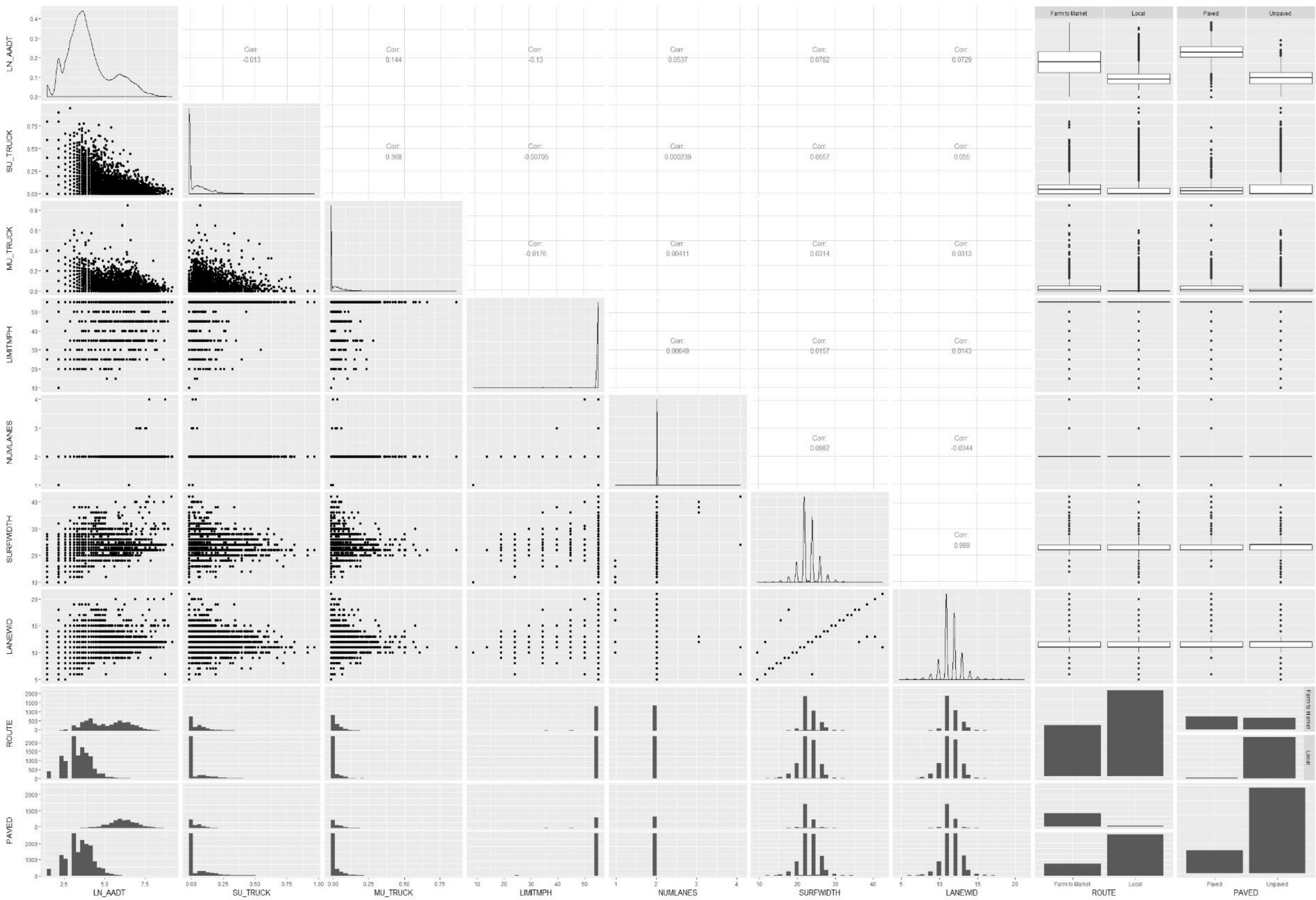


Figure 30: Pearson's correlation analysis results of Traffic Operations Variables

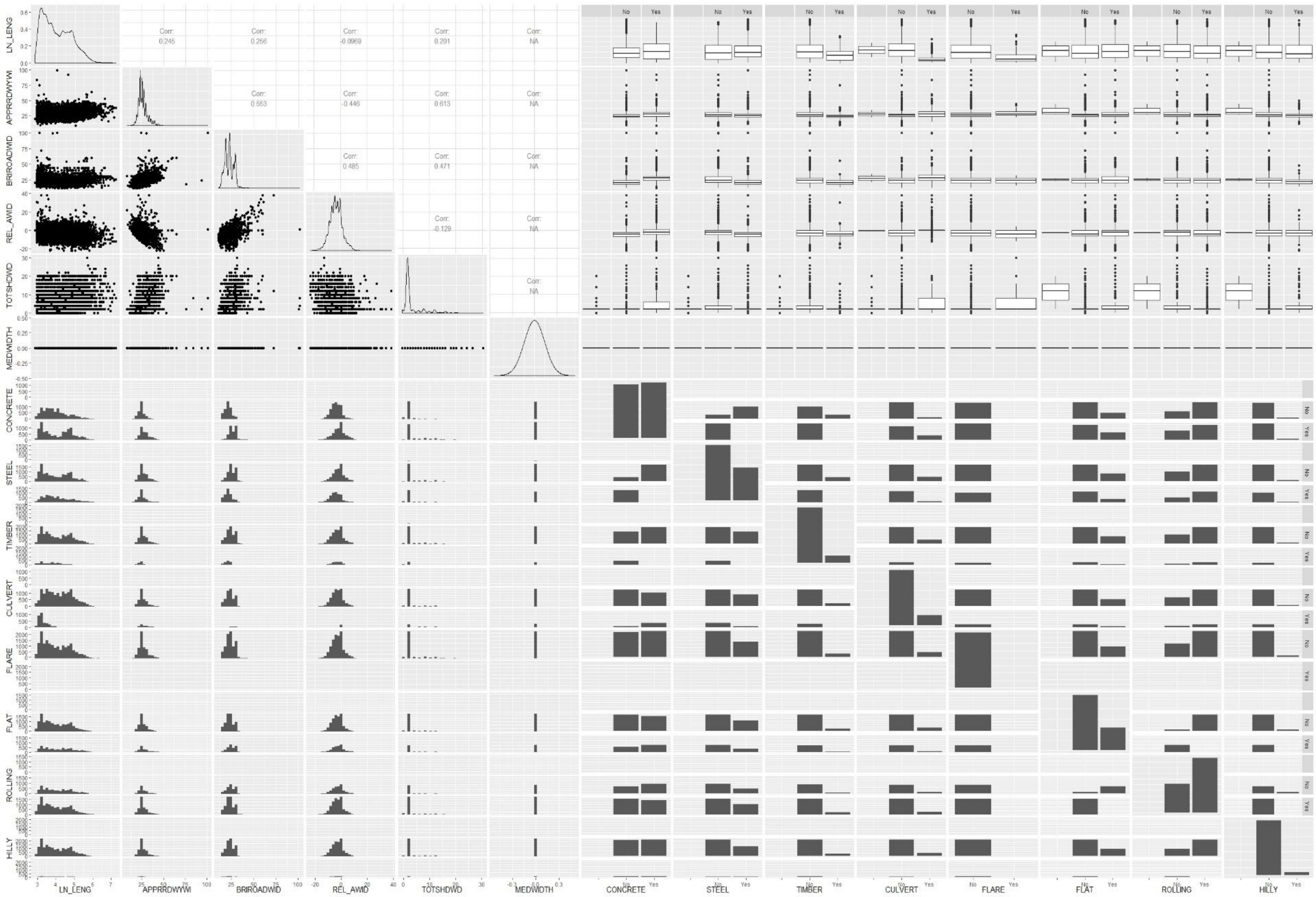


Figure 31: Pearson's correlation analysis results of Bridge Design Variables

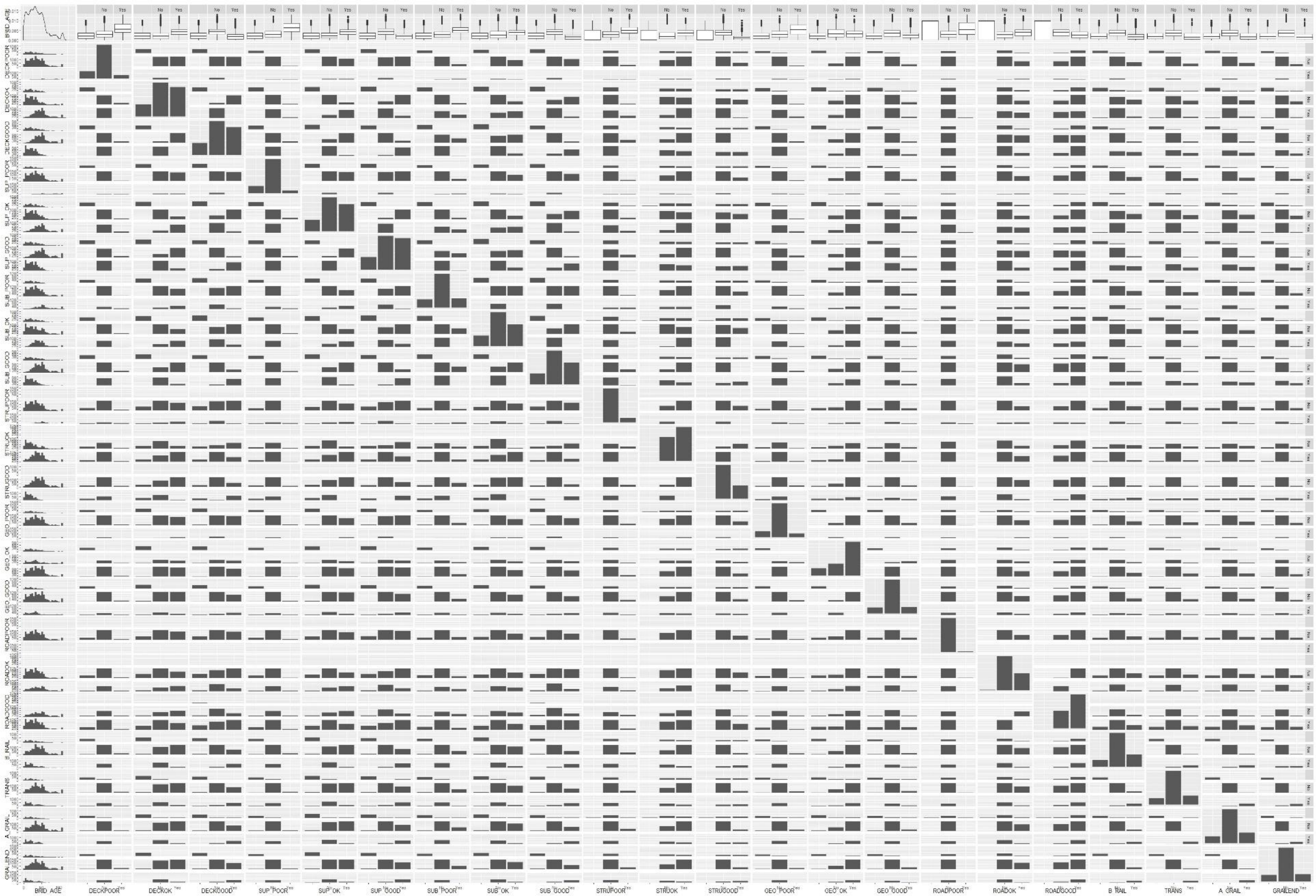


Figure 32: Pearson's correlation analysis results of Bridge Condition Variables

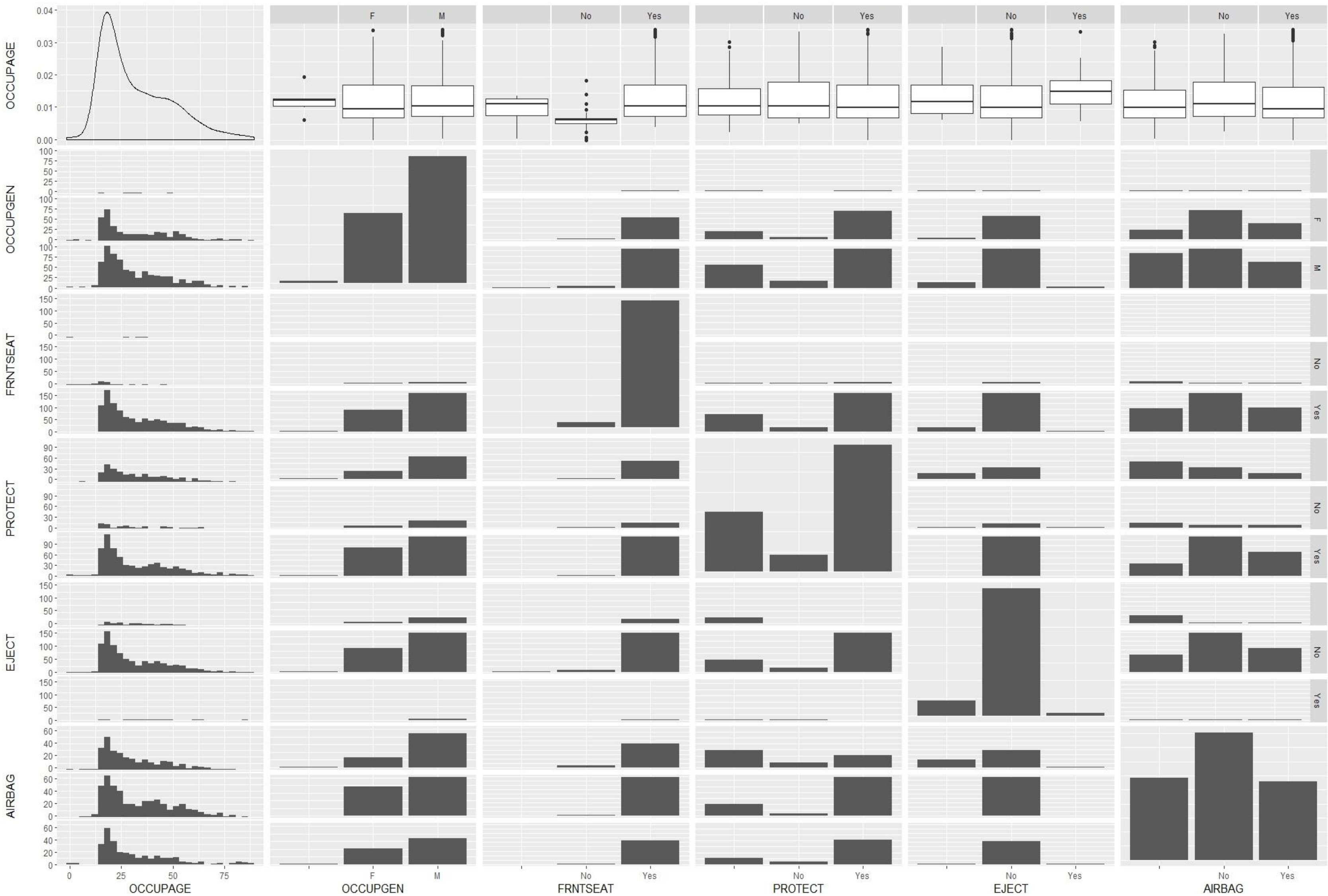


Figure 33: Pearson's correlation analysis results of Person-Level Variables

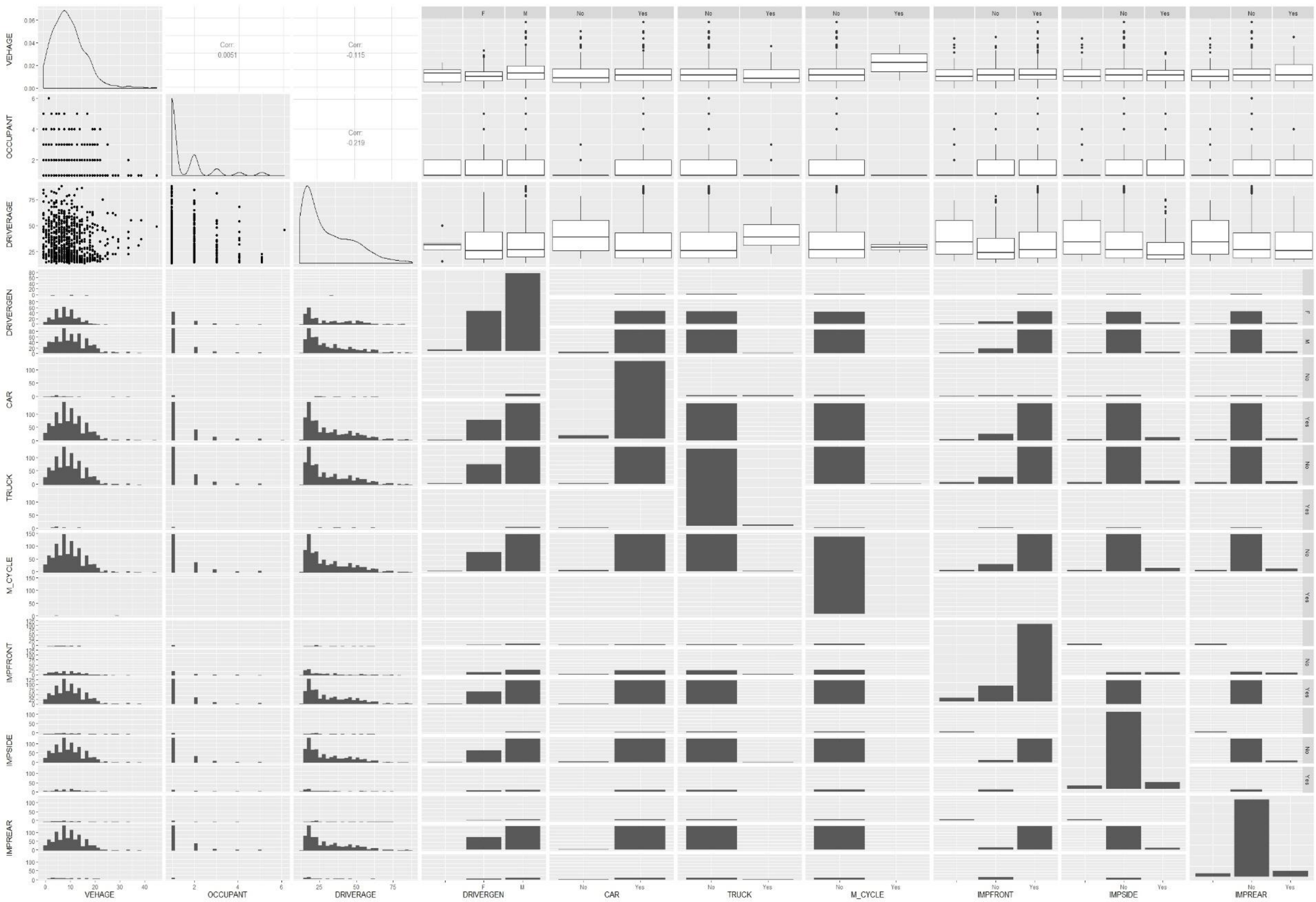


Figure 34: Pearson's correlation analysis results of Vehicle-Level Variables

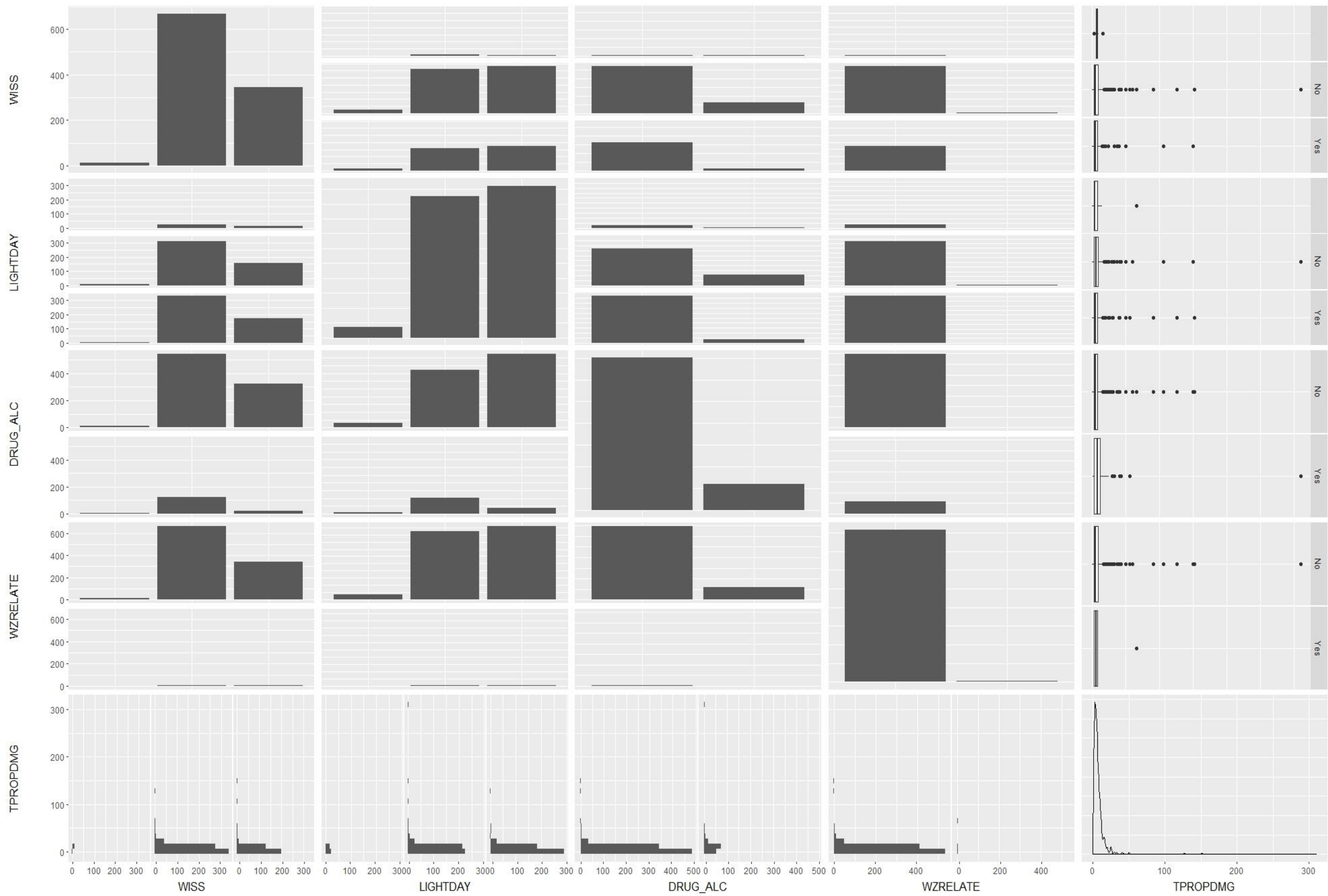


Figure 35: Pearson's correlation analysis results of Crash-Level Variables

Table 20: Variable clusters in the crash frequency model

Cluster	Variables	Classification	# of groups
Route	farm-to-market/local	Traffic Operation	2
Paved Road	yes/no	Traffic Operation	2
Bridge type	concrete/steel/timber	Bridge Design	3
Culvert	yes/no	Bridge Design	2
Flared	yes/no	Bridge Design	2
Terrain	flat/rolling/hilly	Bridge Design	3
Superstructure members	poor/satisfactory/good	Bridge Condition	3
Structure condition	poor/satisfactory/good	Bridge Condition	3
Deck condition	poor/satisfactory/good	Bridge Condition	3
Deck geometry	poor/satisfactory/good	Bridge Condition	3
Roadway alignment	poor/satisfactory/good	Bridge Condition	3
Bridge rail safety	meet standard (yes/no)	Bridge Condition	2
Transitions safety	meet standard (yes/no)	Bridge Condition	2
Approach Guardrail safety	meet standard (yes/no)	Bridge Condition	2
Guardrail ends safety	meet standard (yes/no)	Bridge Condition	2
Total Possible	2x2x3x2x2x3x3x3x3x3x3x2x2x2x2 =		559,872

Table 21: Variable clusters in the injury severity model

Cluster	Variables	Classification	# of groups
Occupant Gender	female/male	Person Level	2
Protection Use	yes/no	Person Level	2
Seating	front/rear	Person Level	2
Ejected	yes/no	Person Level	2
Airbag Deploy	yes/no	Person Level	2
Vehicle Configuration	car/truck/motorcycle	Vehicle Level	3
Driver Gender	female/male	Vehicle Level	2
Initial Impact	front/side/rear	Vehicle Level	3
Surface condition	dry/wet, ice, snow, slush	Crash Level	2
Visibility	day/night, twilight	Crash Level	2
Drug/Alcohol related	yes/no	Crash Level	2
Work zone related	yes/no	Crash Level	2
Total Possible	2x2x2x2x2x3x2x3x2x2x2x2 =		9,216

Table 22: The average marginal effects of the random effects ordered probability model results of occupant injury severity (Dependent variable responses are intergers between 1 [No injury] and 5 [Fatal injury])

Explanatory Variable	Unit Change in Variable	Average Marginal Effects				
		No Injury (PDO)	Possible/ Unknown Injury	Minor Injury	Major Injury	Fatal Injury
OCCU_AGE	1	-0.00316**	0.00061**	0.00175**	0.00062**	0.00018*
PROTECT	Yes	0.33671***	0.01740	-0.17762***	-0.11750***	-0.05899**
FRNTSEAT	Yes	0.21386*	-0.00659	-0.11941**	-0.06246	-0.02540
EJECT	Yes	-0.32303**	-0.02539	0.16851***	0.11832	0.06159
AIRBAG	Yes	-0.18196***	0.02959***	0.10099***	0.03887***	0.01251**
TRUCK	Yes	0.35887**	-0.14250*	-0.16955***	-0.03827***	-0.00854**
OCCUPANT	1	-0.03607	0.00691	0.01997	0.00708	0.00211
IMPFRONT	Yes	0.19528***	-0.01805***	-0.10953***	-0.04962**	-0.01809*
WISS	Yes	0.12304***	-0.02618**	-0.06735***	-0.02288**	-0.00663**
DRUG_ALC	Yes	-0.19413***	0.01551**	0.10895***	0.05077**	0.01889
WZRELATE	Yes	0.29480	-0.10738	-0.14458*	-0.03483***	-0.00801**

Table 23: The elasticities of the random effects ordered probability model results of occupant injury severity (Dependent variable responses are intergers between 1 [No injury] and 5 [Fatal injury])

Explanatory Variable	Percent Change in Variable	Elasticities				
		No Injury (PDO)	Possible/ Unknown Injury	Minor Injury	Major Injury	Fatal Injury
OCCU_AGE	1	-0.16360	0.10418	0.42407	0.87497	1.30278
PROTECT	Yes	0.52476	0.09015	-1.29591	-4.98824	-12.54051
FRNTSEAT	Yes	0.33330	-0.03415	-0.87120	-2.65154	-5.40023
EJECT	Yes	-0.50344	-0.13153	1.22945	5.02318	13.09216
AIRBAG	Yes	-0.28358	0.15331	0.73682	1.65019	2.65872
TRUCK	Yes	0.55929	-0.73825	-1.23705	-1.62463	-1.81599
OCCUPANT	1	-0.08204	0.05224	0.21267	0.43878	0.65332
IMPFRONT	Yes	0.30434	-0.09353	-0.79911	-2.10634	-3.84457
WISS	Yes	0.19175	-0.13561	-0.49139	-0.97117	-1.40992
DRUG_ALC	Yes	-0.30254	0.08034	0.79492	2.15542	4.01629
WZRELATE	Yes	0.45944	-0.55630	-1.05484	-1.47848	-1.70302

```
|-> skip;
|-> negbin;
    lhs=CRASHES;
    rhs=one$
-----------------------------------------------------------------------------
Poisson Regression
Dependent variable              CRASHES
Log likelihood function     -3609.31803
Estimation based on N =  18577, K =   1
Inf.Cr.AIC  =   7220.6 AIC/N =     .389
Model estimated: Jun 20, 2016, 19:29:39
Chi- squared = 25095.01723  RsqP= .0000
G  - squared =  5680.87618  RsqD= .0000
Overdispersion tests: g=mu(i)  :  5.464
Overdispersion tests: g=mu(i)^2:  5.464
--------+--------------------------------------------------------------------
        |                  Standard            Prob.      95% Confidence
 CRASHES|  Coefficient       Error       z    |z|>Z*         Interval
--------+--------------------------------------------------------------------
Constant|   -3.07159***      .03408   -90.13  .0000    -3.13838  -3.00479
--------+--------------------------------------------------------------------
Note: ***, **, * ==>  Significance at 1%, 5%, 10% level.
-----------------------------------------------------------------------------
Line search at iteration    1 does not improve fn. Exiting optimization.
With < 4 iterations, this may not be a good solution to the
optimization. (The log-likelihood is flat.) Try refitting
with ;Output=3 and examining the derivatives.
-----------------------------------------------------------------------------
Negative Binomial Regression
Dependent variable              CRASHES
Log likelihood function     -3448.45964
Restricted log likelihood   -3609.31803
Chi squared [   1 d.f.]       321.71677
Significance level               .00000
McFadden Pseudo R-squared      .0445675
Estimation based on N =  18577, K =   2
Inf.Cr.AIC  =   6900.9 AIC/N =     .371
Model estimated: Jun 20, 2016, 19:29:40
NegBin form 2; Psi(i) = theta
Tests of Model Restrictions on Neg.Bin.
Model                Logl ChiSquared[df]
Poisson(b=0)    -3609.32  ******** [**]
Poisson         -3609.32        .0 [ 0]
Negative Bin.   -3448.46     321.7 [ 1]
--------+--------------------------------------------------------------------
        |                  Standard            Prob.      95% Confidence
 CRASHES|  Coefficient       Error       z    |z|>Z*         Interval
--------+--------------------------------------------------------------------
Constant|   -3.07159***      .03867   -79.44  .0000    -3.14737  -2.99580
        |Dispersion parameter for count data model
   Alpha|    6.81453***      .69509     9.80  .0000     5.45218   8.17688
--------+--------------------------------------------------------------------
Note: ***, **, * ==>  Significance at 1%, 5%, 10% level.
-----------------------------------------------------------------------------
```

```
|-> skip;
|-> negbin;
    lhs=CRASHES;
    rhs=one,LN_AADT,SM_TRUCK,LN_LENG,REL_AWID,PAVED,TIMBER,STEEL,HILLY,BRID_AG
    E,B_RAIL,SUP_POOR,ROADPOOR;
    rst=b0,1.0,b2,b3,b4,b5,b6,b7,b8,b9,b10,b11,b12,b13;
    marginal effects$

---------------------------------------------------------------
Deleted   3755 observations with missing data. N is now  14822
---------------------------------------------------------------

-----------------------------------------------------------------------------
Poisson Regression
Dependent variable              CRASHES
Log likelihood function     -2561.50628
Restricted log likelihood   -3084.28511
Chi squared [  12 d.f.]      1045.55767
Significance level               .00000
McFadden Pseudo R-squared      .1694976
Estimation based on N =  14822, K =  13
Inf.Cr.AIC  =   5149.0 AIC/N =     .347
Model estimated: Jun 20, 2016, 12:14:25
Chi- squared = 15231.22419  RsqP= .2456
G  - squared =  3786.52417  RsqD= .2164
Overdispersion tests: g=mu(i)  :  3.431
Overdispersion tests: g=mu(i)^2:  4.128
--------+--------------------------------------------------------------------
        |                  Standard            Prob.      95% Confidence
 CRASHES|  Coefficient       Error       z    |z|>Z*         Interval
--------+--------------------------------------------------------------------
Constant|   -8.11938***      .28280   -28.71  .0000    -8.67366  -7.56510
 LN_AADT|     .76173***      .04459    17.08  .0000      .67434    .84912
SM_TRUCK|    -.91620         .85014    -1.08  .2812    -2.58244    .75004
 LN_LENG|     .25769***      .05172     4.98  .0000      .15632    .35905
REL_AWID|    -.02607***      .00731    -3.57  .0004     -.04040   -.01174
   PAVED|    -.30546**       .14033    -2.18  .0295     -.58051   -.03042
  TIMBER|     .42048***      .15774     2.67  .0077      .11132    .72964
   STEEL|     .16451*        .08480     1.94  .0524     -.00170    .33071
   HILLY|     .21325         .14385     1.48  .1382     -.06870    .49519
BRID_AGE|     .00534***      .00196     2.73  .0064      .00150    .00917
  B_RAIL|     .08197         .08725      .94  .3474     -.08903    .25297
SUP_POOR|     .33558***      .12985     2.58  .0098      .08109    .59008
ROADPOOR|    1.13388***      .20606     5.50  .0000      .73002   1.53774
--------+--------------------------------------------------------------------
Note: ***, **, * ==>  Significance at 1%, 5%, 10% level.
-----------------------------------------------------------------------------

Normal exit:  20 iterations. Status=0, F=    2529.073
```

```
--------------------------------------------------------------------------------
Negative Binomial Regression
Dependent variable              CRASHES
Log likelihood function     -2529.07291
Restricted log likelihood   -2561.50628
Chi squared [   1 d.f.]        64.86674
Significance level               .00000
McFadden Pseudo R-squared      .0126618
Estimation based on N =  14822, K =  13
Inf.Cr.AIC  =   5084.1 AIC/N =     .343
Model estimated: Jun 20, 2016, 12:14:26
NegBin form 2; Psi(i) = theta
Tests of Model Restrictions on Neg.Bin.
Model                Logl ChiSquared[df]
Poisson(b=0)    -3084.29  ******** [**]
Poisson         -2561.51    1045.6 [12]
Negative Bin.   -2529.07      64.9 [ 1]
--------+-----------------------------------------------------------------------
        |                Standard            Prob.      95% Confidence
 CRASHES|  Coefficient     Error       z    |z|>Z*         Interval
--------+-----------------------------------------------------------------------
Constant|   -8.92975***      .29292   -30.49  .0000    -9.50386  -8.35564
 LN_AADT|       1.0     .....(Fixed Parameter).....
SM_TRUCK|   -1.00022         .95615    -1.05  .2955    -2.87425    .87380
 LN_LENG|     .21001***      .05985     3.51  .0004      .09271    .32732
REL_AWID|    -.02603***      .00839    -3.10  .0019     -.04248   -.00958
   PAVED|    -.84581***      .09798    -8.63  .0000    -1.03785   -.65377
  TIMBER|     .45012***      .16591     2.71  .0067      .12495    .77530
   STEEL|     .17632*        .09926     1.78  .0757     -.01822    .37086
   HILLY|     .25236*        .15310     1.65  .0993     -.04770    .55243
BRID_AGE|     .00597***      .00197     3.03  .0025      .00210    .00983
  B_RAIL|     .08680         .10234      .85  .3963     -.11377    .28738
SUP_POOR|     .38659***      .14009     2.76  .0058      .11202    .66116
ROADPOOR|    1.27677***      .20980     6.09  .0000      .86557   1.68797
        |Dispersion parameter for count data model
   Alpha|    1.58486***      .24807     6.39  .0000     1.09866   2.07107
--------+-----------------------------------------------------------------------
Note: ***, **, * ==>  Significance at 1%, 5%, 10% level.
Fixed parameter ... is constrained to equal the value or
had a nonpositive st.error because of an earlier problem.
--------------------------------------------------------------------------------
```

Partial derivatives of expected val. with
respect to the vector of characteristics.
Effects are averaged over individuals.
Observations used for means are All Obs.
Sample average conditional mean .0534
Scale Factor for Marginal Effects .0534

CRASHES	Partial Effect	Standard Error	z	Prob. |z|>Z*	95% Confidence Interval		
LN_AADT	.05342***	.00228	23.44	.0000	.04895	.05789	
SM_TRUCK	-.05343	.05124	-1.04	.2971	-.15385	.04700	
LN_LENG	.01122***	.00325	3.45	.0006	.00484	.01760	
REL_AWID	-.00139	.08139	-.02	.9864	-.16092	.15814	
PAVED	-.05440	.23122	-.24	.8140	-.50759	.39878	#
TIMBER	.02961	.14469	.20	.8378	-.25398	.31321	#
STEEL	.00957	.04778	.20	.8413	-.08408	.10321	#
HILLY	.01508	.06578	.23	.8187	-.11385	.14400	#
BRID_AGE	.00032	.00819	.04	.9689	-.01573	.01637	
B_RAIL	.00466	.04235	.11	.9123	-.07835	.08768	#
SUP_POOR	.02429	.23367	.10	.9172	-.43369	.48226	#
ROADPOOR	.13395*	.07762	1.73	.0844	-.01818	.28608	#

Partial effect for dummy variable is E[y|x,d=1] - E[y|x,d=0]
Note: ***, **, * ==> Significance at 1%, 5%, 10% level.

```
|-> skip$
|-> ordered;
    lhs=INJSTAT;
    rhs=ONE;
    cluster=CRASH_KE; random effects; marginal effects$
Normal exit:   7 iterations. Status=0, F=    1279.529
+----------------------------------------------------------------------+
| Covariance matrix for the model is adjusted for data clustering.     |
| Sample of   1024 observations contained    912 clusters defined by   |
| variable CRASH_KE which identifies by a value a cluster ID.          |
+----------------------------------------------------------------------+

-----------------------------------------------------------------------------
Ordered Probability Model
Dependent variable              INJSTAT
Log likelihood function     -1279.52948
Estimation based on N =   1024, K =   4
Inf.Cr.AIC  =   2567.1 AIC/N =    2.507
Model estimated: Jun 21, 2016, 17:16:06
Underlying probabilities based on Normal
--------+--------------------------------------------------------------------
        |                  Standard            Prob.      95% Confidence
 INJSTAT|  Coefficient       Error       z    |z|>Z*         Interval
--------+--------------------------------------------------------------------
        |Index function for probability
Constant|    -.00734         .03998     -.18  .8543     -.08570    .07102
        |Threshold parameters for index
  Mu(01)|     .62174***      .05457    11.39  .0000      .51479    .72868
  Mu(02)|    1.38419***      .10707    12.93  .0000     1.17433   1.59405
  Mu(03)|    1.98008***      .15697    12.61  .0000     1.67243   2.28774
--------+--------------------------------------------------------------------
Note: ***, **, * ==>  Significance at 1%, 5%, 10% level.
-----------------------------------------------------------------------------

+---------------------------------------------------------------------+
|                CELL FREQUENCIES FOR ORDERED CHOICES                 |
+---------------------------------------------------------------------+
|               Frequency        Cumulative  < =    Cumulative  > =  |
|Outcome        Count    Percent   Count    Percent   Count    Percent |
|----------- ------- ---------  ------- ---------  ------- --------- |
|INJSTAT=00      515   50.2930      515   50.2930     1024  100.0000 |
|INJSTAT=01      238   23.2422      753   73.5352      509   49.7070 |
|INJSTAT=02      187   18.2617      940   91.7969      271   26.4648 |
|INJSTAT=03       60    5.8594     1000   97.6563       84    8.2031 |
|INJSTAT=04       24    2.3438     1024  100.0000       24    2.3438 |
+---------------------------------------------------------------------+
```

```
-----------------------------------------------------------------------------
Marginal effects for ordered probability model
M.E.s for dummy variables are Pr[y|x=1]-Pr[y|x=0]
Names for dummy variables are marked by *.
--------+--------------------------------------------------------------------
        |     Partial                            Prob.      95% Confidence
 INJSTAT|      Effect    Elasticity      z    |z|>Z*         Interval
--------+--------------------------------------------------------------------
        |--------------[Partial effects on Prob[Y=00] at means]--------------
        |--------------[Partial effects on Prob[Y=01] at means]--------------
        |--------------[Partial effects on Prob[Y=02] at means]--------------
        |--------------[Partial effects on Prob[Y=03] at means]--------------
--------+--------------------------------------------------------------------
z, prob values and confidence intervals are given for the partial effect
Note: ***, **, * ==>  Significance at 1%, 5%, 10% level.
-----------------------------------------------------------------------------


Cross tabulation of predictions and actual outcomes
+------+-----+-----+-----+-----+-----+-----+
|y(i,j)|  0  |  1  |  2  |  3  |  4  |Total|
+------+-----+-----+-----+-----+-----+-----+
|   0  |  515|    0|    0|    0|    0|  515|
|   1  |  238|    0|    0|    0|    0|  238|
|   2  |  187|    0|    0|    0|    0|  187|
|   3  |   60|    0|    0|    0|    0|   60|
|   4  |   24|    0|    0|    0|    0|   24|
+------+-----+-----+-----+-----+-----+-----+
| Total| 1024|    0|    0|    0|    0| 1024|
+------+-----+-----+-----+-----+-----+-----+
Row = actual, Column = Prediction, Model = Probit
Prediction is number of the most probable cell.

Cross tabulation of outcomes and predicted probabilities.
+------+-----+-----+-----+-----+-----+-----+
|y(i,j)|  0  |  1  |  2  |  3  |  4  |Total|
+------+-----+-----+-----+-----+-----+-----+
|   0  |  259|  120|   94|   30|   12|  515|
|   1  |  120|   55|   43|   14|    6|  238|
|   2  |   94|   43|   34|   11|    4|  187|
|   3  |   30|   14|   11|    4|    1|   60|
|   4  |   12|    6|    4|    1|    1|   24|
+------+-----+-----+-----+-----+-----+-----+
| Total|  515|  238|  187|   60|   24| 1024|
+------+-----+-----+-----+-----+-----+-----+
Row = actual, Column = Prediction, Model = Probit
Value(j,m)=Sum(i=1,N)y(i,j)*p(i,m).
Column totals may not match cell sums because of rounding error.
```

```
|-> skip;
|-> ordered;
    lhs=INJSTAT;

rhs=ONE,OCCUPAGE,PROTECT,FRNTSEAT,EJECT,AIRBAG,TRUCK,OCCUPANT,IMPFRONT,WISS,D
RUG_ALC,WZRELATE;
    cluster=CRASH_KE; random effects; marginal effects$

---------------------------------------------------------------
Deleted    205 observations with missing data. N is now    819
---------------------------------------------------------------
Normal exit:  19 iterations. Status=0, F=    623.1442
+-----------------------------------------------------------------------+
| Covariance matrix for the model is adjusted for data clustering.      |
| Sample of    819 observations contained    757 clusters defined by    |
| variable CRASH_KE which identifies by a value a cluster ID.           |
+-----------------------------------------------------------------------+

-----------------------------------------------------------------------------
Ordered Probability Model
Dependent variable              INJSTAT
Log likelihood function      -623.14423
Restricted log likelihood    -693.26859
Chi squared [  11 d.f.]       140.24873
Significance level               .00000
McFadden Pseudo R-squared      .1011504
Estimation based on N =    557, K =  15
Inf.Cr.AIC  =   1276.3 AIC/N =    2.291
Model estimated: Jun 21, 2016, 17:08:00
Underlying probabilities based on Normal
--------+--------------------------------------------------------------------
        |                  Standard            Prob.      95% Confidence
 INJSTAT|  Coefficient       Error       z    |z|>Z*         Interval
--------+--------------------------------------------------------------------
        |Index function for probability
Constant|    1.34286***      .41243     3.26  .0011       .53451   2.15121
OCCUPAGE|     .00792**       .00315     2.51  .0120       .00174    .01410
 PROTECT|    -.94764***      .19768    -4.79  .0000     -1.33508   -.56020
FRNTSEAT|    -.56460*        .33300    -1.70  .0900     -1.21728    .08807
   EJECT|     .92275*        .53066     1.74  .0821      -.11733   1.96283
  AIRBAG|     .46089***      .10669     4.32  .0000       .25178    .66999
   TRUCK|   -1.06028*        .59829    -1.77  .0764     -2.23290    .11234
OCCUPANT|     .09041         .06481     1.39  .1631      -.03663    .21744
IMPFRONT|    -.50261***      .14852    -3.38  .0007      -.79370   -.21151
    WISS|    -.30982***      .11983    -2.59  .0097      -.54468   -.07497
DRUG_ALC|     .50160***      .16607     3.02  .0025       .17610    .82709
WZRELATE|    -.81753         .63993    -1.28  .2014     -2.07178    .43671
        |Threshold parameters for index
  Mu(01)|     .66075***      .06823     9.68  .0000       .52703    .79448
  Mu(02)|    1.66239***      .14072    11.81  .0000      1.38659   1.93818
  Mu(03)|    2.39213***      .20922    11.43  .0000      1.98206   2.80219
--------+--------------------------------------------------------------------
Note: ***, **, * ==>  Significance at 1%, 5%, 10% level.
-----------------------------------------------------------------------------
```

CELL FREQUENCIES FOR ORDERED CHOICES

Outcome	Frequency Count	Frequency Percent	Cumulative < = Count	Cumulative < = Percent	Cumulative > = Count	Cumulative > = Percent
INJSTAT=00	415	50.6716	415	50.6716	819	100.0000
INJSTAT=01	174	21.2454	589	71.9170	404	49.3284
INJSTAT=02	167	20.3907	756	92.3077	230	28.0830
INJSTAT=03	44	5.3724	800	97.6801	63	7.6923
INJSTAT=04	19	2.3199	819	100.0000	19	2.3199

Marginal effects for ordered probability model
M.E.s for dummy variables are Pr[y|x=1]-Pr[y|x=0]
Names for dummy variables are marked by *.

INJSTAT	Partial Effect	Elasticity	z	Prob. |z|>Z*	95% Confidence Interval	
	[Partial effects on Prob[Y=00] at means]					
OCCUPAGE	-.00316**	-.16360	-2.51	.0120	-.00563	-.00069
*PROTECT	.33671***	.52476	5.99	.0000	.22663	.44680
*FRNTSEA	.21386*	.33330	1.88	.0602	-.00916	.43689
*EJECT	-.32303**	-.50344	-2.30	.0212	-.59774	-.04833
*AIRBAG	-.18196***	-.28358	-4.40	.0000	-.26305	-.10088
*TRUCK	.35887**	.55929	2.56	.0103	.08455	.63318
OCCUPANT	-.03607	-.08204	-1.39	.1630	-.08674	.01461
*IMPFRON	.19528***	.30434	3.56	.0004	.08773	.30283
*WISS	.12304***	.19175	2.61	.0090	.03067	.21541
*DRUG_AL	-.19413***	-.30254	-3.21	.0013	-.31273	-.07553
*WZRELAT	.29480	.45944	1.59	.1118	-.06860	.65819
	[Partial effects on Prob[Y=01] at means]					
OCCUPAGE	.00061**	.10418	2.37	.0176	.00011	.00111
*PROTECT	.01740	.09015	.77	.4393	-.02670	.06150
*FRNTSEA	-.00659	-.03415	-.34	.7327	-.04442	.03124
*EJECT	-.02539	-.13153	-.39	.6987	-.15394	.10316
*AIRBAG	.02959***	.15331	3.59	.0003	.01344	.04575
*TRUCK	-.14250*	-.73825	-1.73	.0843	-.30430	.01929
OCCUPANT	.00691	.05224	1.39	.1644	-.00283	.01665
*IMPFRON	-.01805***	-.09353	-2.86	.0043	-.03044	-.00566
*WISS	-.02618**	-.13561	-2.27	.0234	-.04880	-.00355
*DRUG_AL	.01551**	.08034	2.23	.0261	.00185	.02917
*WZRELAT	-.10738	-.55630	-1.10	.2697	-.29806	.08330

```
        |--------------[Partial effects on Prob[Y=02] at means]--------------
OCCUPAGE|     .00175**        .42407     2.48  .0132       .00037    .00313
*PROTECT|    -.17762***    -1.29591     -7.12  .0000     -.22649   -.12875
*FRNTSEA|    -.11941**      -.87120     -2.04  .0414     -.23416   -.00465
  *EJECT|     .16851***    1.22945      3.48  .0005       .07350    .26352
 *AIRBAG|     .10099***     .73682      4.33  .0000       .05528    .14670
  *TRUCK|    -.16955***    -1.23705     -3.29  .0010     -.27054   -.06857
OCCUPANT|     .01997         .21267     1.40  .1607     -.00793    .04787
*IMPFRON|    -.10953***     -.79911     -3.51  .0004     -.17068   -.04838
   *WISS|    -.06735***     -.49139     -2.63  .0084     -.11747   -.01724
*DRUG_AL|     .10895***      .79492     3.20  .0014       .04222    .17568
*WZRELAT|    -.14458*      -1.05484     -1.89  .0583     -.29424    .00509
        |--------------[Partial effects on Prob[Y=03] at means]--------------
OCCUPAGE|     .00062**        .87497     2.17  .0300       .00006    .00118
*PROTECT|    -.11750***    -4.98824     -2.76  .0058     -.20097   -.03403
*FRNTSEA|    -.06246       -2.65154     -1.19  .2355     -.16564    .04073
  *EJECT|     .11832        5.02318      1.31  .1891     -.05828    .29492
 *AIRBAG|     .03887***    1.65019      3.04  .0024       .01379    .06396
  *TRUCK|    -.03827***    -1.62463     -3.45  .0006     -.06003   -.01651
OCCUPANT|     .00708         .43878     1.26  .2063     -.00390    .01806
*IMPFRON|    -.04962**     -2.10634     -2.35  .0189     -.09105   -.00818
   *WISS|    -.02288**      -.97117     -2.24  .0251     -.04289   -.00286
*DRUG_AL|     .05077**      2.15542      2.17  .0299       .00496    .09659
*WZRELAT|    -.03483***    -1.47848     -2.59  .0096     -.06117   -.00848
        |--------------[Partial effects on Prob[Y=04] at means]--------------
OCCUPAGE|     .00018*       1.30278      1.84  .0663     -.00001    .00038
*PROTECT|    -.05899**    -12.54051     -2.10  .0353     -.11393   -.00406
*FRNTSEA|    -.02540       -5.40023     -1.05  .2946     -.07291    .02210
  *EJECT|     .06159       13.09216       .83  .4093     -.08472    .20790
 *AIRBAG|     .01251**      2.65872      2.24  .0248       .00158    .02343
  *TRUCK|    -.00854**     -1.81599     -2.37  .0180     -.01562   -.00147
OCCUPANT|     .00211         .65332     1.29  .1977     -.00110    .00531
*IMPFRON|    -.01809*      -3.84457     -1.89  .0593     -.03688    .00071
   *WISS|    -.00663**     -1.40992     -2.06  .0395     -.01295   -.00032
*DRUG_AL|     .01889        4.01629      1.62  .1058     -.00401    .04179
*WZRELAT|    -.00801**     -1.70302     -2.15  .0315     -.01531   -.00071
--------+-------------------------------------------------------------------
z, prob values and confidence intervals are given for the partial effect
Note: ***, **, * ==>  Significance at 1%, 5%, 10% level.
----------------------------------------------------------------------------
```

Cross tabulation of predictions and actual outcomes

y(i,j)	0	1	2	3	4	Total
0	399	0	16	0	0	415
1	144	0	27	0	3	174
2	110	0	57	0	0	167
3	28	0	13	0	3	44
4	4	0	11	0	4	19
Total	685	0	124	0	10	819

Row = actual, Column = Prediction, Model = Probit
Prediction is number of the most probable cell.

Cross tabulation of outcomes and predicted probabilities.

y(i,j)	0	1	2	3	4	Total
0	245	84	69	14	3	415
1	85	38	38	10	3	174
2	70	38	40	13	6	167
3	14	10	13	4	3	44
4	3	3	6	3	4	19
Total	417	173	166	44	19	819

Row = actual, Column = Prediction, Model = Probit
Value(j,m)=Sum(i=1,N)y(i,j)*p(i,m).
Column totals may not match cell sums because of rounding error.

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