Bridge Curb/Railing and Approach Treatment for Extremely Low Volume Roads

FINAL REPORT

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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LIST OF ACRONYMS

AADT	annual average daily traffic
ADT	average daily traffic
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
B/C	benefit-cost
BCAP	Benefit-Cost Analysis Program
DOT	department of transportation
FEA	finite element analysis
FHWA	Federal Highway Administration
FLMA	Federal Land Management Agency
FLTP	Federal Lands Transportation Program
FMVSS	Federal Motor Vehicle Safety Standards
GSBR	Guide Specifications for Bridge Railings
ICER	incremental cost effectiveness ratio
I.M.	instructional memorandums
ISPE	in-service performance evaluation
KA	fatal and serious injury
LON	length of need
MASH	Manual for Assessing Safety Hardware
MoDOT	Missouri Department of Transportation
mph	miles per hour
MUTCD	Manual on Uniform Traffic Control Devices
MwRSF	Midwest Roadside Safety Facility
NBI	National Bridge Inventory
NCHRP	National Cooperative Highway Research Program
NHS	National Highway System
NHTSA	National Highway Traffic Safety Administration
OIV	occupant impact velocity
ORA	occupant ridedown accelerations
PL1	Performance Level 1
PL2	Performance Level 2
PG	performance goal

RDG	Roadside Design Guide
RRR	relative risk reduction
RSAP	Roadside Safety Analysis Program
SUT	single unit truck
TAC	technical advisory committee
TL1	Test Level 1
TTI	Texas A&M Transportation Institute
UNL	University of Nebraska-Lincoln
USBR	United States Bureau of Reclamation
VLV	very low volume
vpd	vehicles per day
WVBR	West Virginia Timber Curb-Type Bridge Rail

INTRODUCTION

Bridges cross over a variety of features ranging from drainage ditches to deep valleys and serve a variety of functions. There are several competing considerations when evaluating bridge railings: the risk of injury and death for vehicle drivers and occupants, the structural integrity of the bridge itself, and the service provided to the principal users of the bridge.

The objective of this research was to develop a guide for bridge rails and approach treatments for extremely low-volume roads and low-speed roads. This report describes the background research and information used to develop the resulting guide: *Guide for Bridge Curb/Rail and Approach Treatment for Extremely Low Volume Roads* (the Guide).

The Guide is applicable to both single- and two-lane bridges. For the purpose of the Guide, extremely low-volume roads were defined as roads with less than 50 vehicles per day (vpd) and posted speed ranges equal to 5-15 miles per hour (mph), 16-30 mph, and 31-45 mph. The Guide assesses the need for and practicality of providing bridge rails to contain passenger vehicles while accommodating other vehicle types like agricultural equipment, timber harvesting equipment, recreational vehicles, and other associated service vehicles on the subject bridges.

The Guide leads inspectors and engineers to practical, low-cost solutions based on the characteristics of the bridge, anticipated traffic, and the intended use of the bridge. The outcome of using the Guide is the determination of whether the existing bridge rails, associated roadside hardware, and other conditions can be left as-is; when improvements should be considered to achieve the safety performance goal; and what potential solutions might be implemented to achieve the safety performance goal. When new hardware is recommended, the Guide provides construction details for the new hardware. The Guide also includes an inspection checklist that inspectors can use to collect information from an inspection. Evaluating the need to provide roadside barriers, such as w-beam for shielding obstacles on the roadside, beyond the approaches to the bridge, was beyond the scope of this project and the Guide.

This report documents the literature and policy search and the development of parameters for the use of bridge rails, associated hardware and delineation on the extremely low volume and low speed roads. This report also documents the research conducted to modify the Manual for Assessing Safety Hardware (MASH) Test Level 1 (TL-1) low profile West Virginia Timber Curb-Type Bridge Rail (WVBR) for installation on a lower strength bridge structure (e.g., 4 ft by 12 ft wood plank bridge deck).

Multiple concepts were considered for the Guide, including a variety of alternatives for format and content, however, only the final concept is documented herein. This research also included the compilation of drawings and technical information for a variety of bridge rails, transitions, and terminals. The Guide contains the associated drawings.

LITERATURE AND POLICY SEARCH

The United States Bureau of Reclamation (USBR) is a Federal Land Management Agency (FLMA). The Federal Lands Transportation Program (FLTP) was established in 23 U.S.C. 203 to improve the transportation infrastructure owned and maintained by FLMAs. FLTP is one potential funding source for improvement projects for the 315 bridges within the National Bridge

Inventory (NBI) for which USBR is responsible. This USBR inventory is dominated by low-speed and low-volume bridges.

Although low-volume roads make up approximately 80 percent of roadway mileage in the nation's transportation system, these low-volume roads only represent approximately 20 percent of the vehicle miles traveled.(1) Low-volume roads, however, "have a fairly high bridge density, averaging approximately 9 bridges every 100-centerline kilometers (14 bridges every 100-centerline miles."(2)

There has been interest for several decades in developing selection guidance for the multiple performance, service, or test levels of bridge rails. The recent completion of the National Cooperative Highway Research Program (NCHRP) 22-12(03) (*Recommended Guidelines for the Selection of Test Levels 2 through 5 Bridge Rails*) provided this guidance, but did not address extremely low-volume roads nor low-speed roads. (3)

Design of Bridge Rails

Historically, design of bridge rails has followed guidance contained in the American Association of State Highway Officials (AASHO) *Standard Specifications for Highway Bridges*. Prior to 1965, the AASHO specification required very simply that "substantial railings along each side of the bridge shall be provided for the protection of traffic."(4, 5) It was specified that the top members of bridge railings be designed to simultaneously resist a lateral horizontal force of 150 lb/ft and a vertical force of 100 lb/ft applied at the top of the railing. The design load on lower rail members varied inversely with curb height, ranging from 500 lb/ft for no curb to 300 lb/ft for curb heights of 9 inches or greater. It was further specified that the railing have a minimum height of 27 inches and a maximum height of 42 inches above the roadway surface.(4) These loads are only a fraction of what is used today.

Revised railing specifications were subsequently published in 1965 in the 9th edition of the AASHO *Standard Specifications for Highway Bridges*.(6) It required that rails and parapets be designed for a transverse load of 10,000 lbs divided among the various rail members using an elastic analysis. The force was applied as a concentrated load at the mid-span of a rail panel with the height and distribution of the load based on rail type and geometry. The height of the railing was required to be no less than 27 inches and railing configurations successfully crash tested were exempt from the design provisions.(6)

Olson was perhaps the first to systematically examine the performance requirements for bridge railings in 1970. His results, documented in NCHRP Report 86, suggested use of appropriate transitions, evaluation through crash testing, ability to minimize bridge rail penetration, and other things that are considered standard objectives of bridge railing design today.(7)

Crash testing is the most direct means of assessing barrier impact performance. Full-scale crash testing is typically used to verify the ability of a barrier to contain an impacting vehicle. The traffic volumes, geometrics, and operational characteristics, which can affect the likelihood of a crash, are not addressed by crash testing. The next section summarizes the different performance and test levels that have been used over the years to evaluate crash test performance. A section on guidelines for when to use barriers follows.

AASHTO Guide Specification for Bridge Railings

In 1989, AASHTO published the *Guide Specifications for Bridge Railings* (GSBR) to provide a more comprehensive approach for the design, testing, and selection of bridge rails than that contained in the AASHTO *Standard Specifications for Highway Bridges*.[8, 9] The GSBR emphasized the need for full-scale crash testing to verify that a given bridge rail design meets the desired impact performance criteria. The document defined three bridge rail performance levels and associated crash tests to be used in qualifying the railing. The crash test matrices for each performance level were described by crash test conditions defined in terms of vehicle type, vehicle weight, impact speed, and impact angle. Two passenger vehicles—a small 1,800-lb passenger car and a 5,400-lb pickup truck—were common to all three performance levels. Test conditions associated with Performance Level 1 (PL1) included an 1,800-lb small car and a 5,400-lb pickup truck impacting at an angle of 20 degrees and a speed of 50 mph and 45 mph, respectively. For Performance Level 2 (PL2), the speed of the small car and pickup truck tests were increased to 60 mph.

Although the 1989 GSBR recommended crash testing as the basis for bridge rail evaluation and acceptance, it did provide the bridge engineer with suggested design information including the magnitude, distribution, and vertical location of railing design loads for each performance level. The transverse loads were derived from two related research studies in which vehicle impact forces were measured using instrumented concrete walls.[10, 11]

NCHRP Report 350

In 1993, NCHRP Report 350 was published, superseding the previous crash testing guidelines contained in NCHRP Report 230.[12] One major change in Report 350 is that six different test levels for roadside hardware were added for bridge railings. The intent was to provide test guidelines for developing a range of longitudinal barriers, including bridge railings and transitions, that could be used in different situations. TLs 1 through 3 relate to containment of passenger vehicles (e.g., small passenger cars and pickup trucks) and vary by impact speed, with increasing impact speeds defined for increasing test levels: 31 mi/hr for TL-1, 44 mi/hr for TL-2, and 62 mi/hr for TL-3.) While Report 350 provided the testing recommendations, only general guidance was provided about what field conditions would indicate the need for a particular test level bridge railing.

The Federal Highway Administration (FHWA) established approximate equivalences between the multiple service levels, performance levels, and test levels in a memorandum to FHWA Regional Administrators in 1997.[13] The equivalencies set out by FHWA are summarized in table 1.

Table 1. Crash test acceptance equivalencies from FHWA.[13]

Bridge Railing Testing Criteria	Acceptance Equivalencies					
Report 350	TL1	TL2	TL3	TL4	TL5	TL6
Report 230		MSL-1 MSL-2†				
AASHTO Guide Spec		PL1		PL2	PL3	
AASHTO LRFD Bridge Spec		PL1		PL2	PL3	

† This is the performance level usually cited when describing a barrier tested under NCHRP Report 230. It is close to TL3 but adequate TL3 performance cannot be assured without a pickup truck test.

Manual for Assessing Safety Hardware

Since the publication of NCHRP Report 350 in 1993, changes have occurred in vehicle fleet characteristics, operating conditions, and technology. NCHRP Project 22-14(2), *Improvement of Procedures for the Safety-Performance Evaluation of Roadside Features*, was initiated to take the next step in the continued evolution of roadside safety testing and evaluation. The results of this research effort culminated in the 2009 AASHTO MASH.[14] MASH was updated again in 2016.[15]

MASH includes essentially the same test level approach with some changes to the impact conditions for the higher test level longitudinal barrier tests and the vehicles used for testing. Aside from these changes to the test vehicles, TL1 and TL2 remained largely the same as they were in NCHRP 350.

The weight and body style of the pickup truck test vehicle changed from a 2,000 kg (4,409 lb), ¾-ton, standard cab pickup to a 2,270 kg (5,000 lb), ½-ton, 4-door pickup. This change in vehicle mass of approximately 15 percent was deemed to produce an impact condition that was similar to, and possibly more severe than the TL4 single unit truck (SUT) test from NCHRP Report 350.

Crash testing conducted at the Midwest Roadside Safety Facility (MwRSF) at the University of Nebraska-Lincoln (UNL) under NCHRP Project 22-14(2) and the Texas A&M Transportation Institute (TTI) under NCHRP Project 22-14(3) demonstrated that taller barriers will be required. Dobrovolny et al. recently developed MASH equivalencies of NCHRP Report 350 bridge railings.[16] Ray and Carrigan considered the MASH equivalencies in NCHRP Web Only Document 307, *Recommended Guidelines for the Selection of Test Levels 2 Through 5 Bridge Rails*.[17] The equivalency results from these research efforts and the previously discussed table 1 Report 350 equivalencies are summarized in table 2.

Table 2. Approximate crash test acceptance equivalencies.[18, 19]

Bridge Railing Testing Criteria	Acceptance Equivalencies					
MASH	TL1	TL2	TL3	TL4	TL5	TL6
Report 350 • Solid Concrete • Metal beam and post (on ≥24 in concrete parapet)		TL2	TL3 & TL4		TL5	
Report 350 • Concrete Beam and Post • Metal Beam and Post (deck mounted) • Metal Beam and Post (on curb)		TL2, TL3, & TL4			TL5	
Report 230		MSL-1 MSL-2†				
AASHTO Guide Spec		PL1	PL2		PL3	
AASHTO LRFD Bridge Spec		PL1	PL2		PL3	

† This is the performance level usually cited when describing a barrier tested under NCHRP Report 230. It is close to TL3 but adequate TL3 performance cannot be assured without a pickup truck test.

Longitudinal Barriers for Passenger Vehicle Containment

While most of the design and testing effort under Report 350 and MASH involved designing higher test level bridge railings, a number of TL1 and 2 bridge railings were also developed. Faller, for example, designed and tested three TL1 bridge railings for timber bridge decks as well as three curb railings for very low speed (i.e., 15 mi/hr) applications.[20, 21] These and other plans for timber rails on concrete decks were subsequently documented in the USDA *Forest Products Laboratory* plan sets.[22]

A number of bridge rail, transitions, and terminals were found in the literature which have been crash tested to demonstrate passenger vehicle containment at low-impact speeds. In addition to these systems, the literature also includes suggestions for using long-span guardrail on short bridges and small radius guardrail on approaches. Appendix A of the Guide provides a culmination of the hardware found through the literature and developed under this research effort. Construction drawings and associated technical details to constructing this hardware were developed and are summarized in the Guide appendices along with the source materials.

Guidelines and Specifications

FHWA and AASHTO

AASHTO Guide Specification for Bridge Railings

Engineers have recognized that bridge rail needs can vary significantly from site to site based on a multitude of factors. For the first time, the 1989 AASHTO *GSBR* provided bridge engineers guidance for determining the appropriate railing performance level for a given bridge site.(8) Selection tables were provided that estimated the appropriate railing performance level for a given bridge site based on highway and site characteristics like highway type (e.g., divided, undivided, etc.), design speed, traffic volume, percent trucks, and bridge rail offset. The tables applied to bridges that were on tangent, level roadways with deck surfaces approximately 35-ft above the underlying ground or water surface. It was further assumed that there was low occupancy land use or shallow water under the bridge structure. Correction factors were provided to permit the engineer to adjust the traffic volume for horizontal curvature, vertical grade, different deck heights, and different densities of land use beneath the bridge. These selection procedures were developed using a benefit-cost (B/C) analysis combined with engineering judgment. While much thought and effort went into the derivation of the bridge rail section tables and performance levels, an independent review of the bridge rail performance selection guidelines found that some of the data and assumptions used in the B/C analysis were faulty.(23) Consequently, the selection guidelines were never implemented on a national basis. The 1989 *GSBR* did not address lower volume, lower speed bridge railings.(8)

AASHTO Roadside Design Guide

The AASHTO *Roadside Design Guide* (RDG) was first published in 1989, the most recent, fourth edition, was published in 2011.(24, 25) Chapter 7 of the RDG covers the selection and placement of bridge railings and transitions. Most of the RDG is dedicated to guidance regarding high-speed and/or high-volume roadways; however, some guidance is provided for low-speed, low-volume roadways. The 2011 RDG states "full treatment may not be cost-effective on bridge-length culverts, and alternate treatments should be considered. Such treatments could include extending the structure and leaving the edges unshielded or using a less expensive, semi-rigid type railing."(24) Additionally, Chapter 12 of the RDG covers guidance for low-volume roads and streets. In this chapter the guidance is tailored to low-cost countermeasures. For this reason, signage and delineation is discussed as the most cost-beneficial improvement to low-volume, low-speed roads. The 2011 RDG guidance for bridge railings on low-volume roads includes a discussion of approach railings. The RDG observes that approach railing, when used, should not direct an impacting vehicle into the parapet. The RDG also provides that "improving a deficient [bridge] railing by extending the approach guardrail completely across the structure is often used and very effective countermeasure. Although such a modified design may not meet full design standards, it will provide continuity of the rail if struck by errant motorists."(24)

NCHRP Project 22-12(03)

While a variety of bridge railings have been developed at various test levels, what had not been established were the criteria for selecting which test level should be used based on specific traffic and site conditions. The AASHTO RDG recognizes the multiple test level approach but gives only very general guidance about why different test level bridge railing might be used.(24) At present, highway agencies must make decisions on which test level is appropriate for each site based on an *ad hoc* basis. The objective of NCHRP Project 22-12(03), *Recommended Guidelines for the Selection of Test Levels 2 through 5 Bridge Rails*, was to develop such guidelines.(18) The conditions where TL2 through TL5 bridge railings should be considered were examined in that research. A selection procedure was developed that was based on the risk of a vehicle penetrating bridge railing and resulting in a fatal or serious injury crash. The procedure accounts for the traffic volume, posted speed, land use under the bridge, and road characteristics (e.g., grade, curvature, lane width, etc.). The traffic and site characteristics are quantified by the designer and used to predict the risk of a fatal or serious injury crash. NCHRP Project 22-12(03) was limited to TL2 through TL5 bridge railings and addressed neither extremely low-volume roads nor low-speed roads.

AASHTO Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT ≤ 400)

AASHTO's *Guidelines for Geometric Design of Very Low-Volume Roads* (VLV Guide) observes "roadside design is the one major determinant of safety on very low-volume local roads." (26) The VLV Guide also observes, however, that "roadside clear zones provide very little benefit, and that traffic barriers are not generally cost-effective, on roads with very low traffic volumes." In addition, the VLV Guide indicates that "the roadside design guidelines for very low-volume local roads provide great flexibility to the designer in exercising engineering judgement to decide where it is appropriate to provide improved roadsides."(26)

FLH Barrier Guide for Low Volume and Low Speed Roads

The 2005 *FLH Barrier Guide* defines low-volume roads as those roads with an average daily traffic (ADT) of less than 2,000 vehicles per day vpd. The *FLH Barrier Guide* outlines challenges faced by roadway engineers when designing low-volume roads:(2)

- The fatal crash rate is estimated to be three times higher on rural minor roads.
- These roads typically have more restricted rights-of-way, little or no clear zones, and substandard design features.
- High bridge density and issues associated with bridges including restricted conditions and rigid rails.
- Corrective measures can be difficult to justify economically.

The *FLH Barrier Guide* observes that low travel speeds and driver familiarity can mitigate some of the challenges mentioned above, however, fatal and serious injury crashes can and do still occur on low-speed roads.

Locating and designing barrier systems, as well as properly designing and locating approach rail/transition sections are covered in Chapter 4 of the *FLH Barrier Guide*. The design method outlined in the RDG is the chosen method for locating longitudinal barriers (including approach rail). Values for determining the length of need (LON) presented in the *FLH Barrier Guide* cover lower speeds and lower traffic volumes than what is published in the 2002 RDG.(2, 27) It is not clear where these values for lower speed and lower traffic volume came from.

Bureau of Reclamation Guidelines for Determining the Adequacy of Existing Bridge Safety Features at Reclamation's Bridges

The 2006 Bureau of Reclamation *Guidelines for Determining the Adequacy of Existing Bridge Safety Features at Reclamation's Bridges* is used in combination with the Bridge Inventory and Inspection Program Directives and Standards (*Reclamation Manual Directives and Standards*

FAC 07-01) "with the purpose of aiding Reclamation's bridge inspectors in determining the adequacy of bridge traffic safety features on Reclamation's bridges."(28)

This Guide classifies bridges into two categories. "Class A bridges (Low-Speed, Low-Volume) are those bridges which meet all of the following:

- With an approximate ADT volume (estimated at time of inspection, if no traffic counts are available) less than or equal to 50 vpd,
- And with estimated traffic speeds approaching and across the bridge of 15 mph or less, and
- The structure is a standard width one lane bridge (i.e., 16 ft or less).

Class B bridges are those bridges (greater than Class A), that do not meet the criteria for a Class A bridge."(28) Bridges are further divided by type where Type 1 bridges are open to the public and Type 2 bridges are not. Recommendations are made regarding bridge rail and approach rails based on the class and type of the bridge.

It is suggested within the guidance that Type 1, Class A bridges should have, at a minimum, a curb-type rail (crash tested or equivalent). Other factors where a higher level of protection might be warranted for Type 1, Class A bridges are listed as follows:

- "Historical data, including accident history at the site.
- Approximate vehicle speed approaching and across the bridge.
- Approximate traffic volume.
- One-way versus two-way traffic.
- Number of traffic lanes.
- Type of vehicles using the bridge, i.e. trucks, RVs, etc.
- Bridge surface (i.e., slippery vs. non-slippery).
- Approach roadway vertical and horizontal alignment.
- Approach roadway width vs. bridge clear deck width.
- Approach roadway surface (i.e., gravel, pavement, etc.).
- Approach roadway embankment steepness.
- Bridge features (i.e., length, width, deck crown, etc.).
- Waterway features (i.e., height of bridge above waterway, depth of water, etc.).
- Typical weather conditions (i.e., snow, ice, high winds, fog, etc.).
- Nighttime use.
- Familiarity of users with bridge (i.e., local traffic only, tourists, etc.).
- Probable severity of injury from a bridge run-off due to bridge height, depth of water, vehicle speed, etc.
- Pedestrian use.

- Projected future use of the bridge.
- Aesthetic concerns."

Further guidance is provided for approach rails and terminals for these classes and types of bridges.

State Departments of Transportation

Many State DOTs refer designers to Chapter 13 of the AASHTO *LRFD Bridge Design Specifications* for bridge rail strength and geometric requirements, NCHRP 350 for crash test criteria, and the AASHTO *Guide Specification* and RDG for guidance on design and installation of bridge rail and approach guardrail. These national documents are supplemented in many States with State-specific guidance in which the States detail the use of specific railings under certain situations. A sample of State-adopted guidance for bridges carrying low-volume roads is presented in this section.

Illinois Department of Transportation

Chapter 35 of the Illinois Department of Transportation (DOT) published *Bureau of Local Roads and Streets Manual* allows for the elimination of approach rail on the bridge rail end closest to traffic if one or more of the following apply:

- "The posted speed limit is less than 25 mph on an urban curbed section.
- The ADT is less than 150; the bridge is approach roadway width; and the bridge is on tangent alignment.
- A township or road district bridge is wider than the approaching roadway and the bridge is on tangent alignment."(29)

Iowa DOT

The Iowa DOT publishes instructional memorandums (I.M.) to assist local public agencies with transportation-related topics. I.M. 3.213 provided guidance for determining the need for traffic barriers at roadway bridges and culverts through 2016. Bridge rail upgrade needs were determined using a point system that assigns points for various characteristics for the bridge such that if the sum of points is less than 25, the bridge rail need not be updated. The categories that are used to assess points include: crash history, annual average daily traffic (AADT), bridge width, bridge length, and existing bridge rail type. The guidance limited the use of approach guardrail "if the following conditions exist:

- AADT is less than 400 vpd,
- The structure is at least 24-ft wide,
- The structure is on a tangent road section,
- The B/C ratio is less than 0.8, and
- The bridge width is wider than the approach roadway width."(30)

In 2018, substantive changes were made to I.M. 3.213 and it was re-numbered to I.M. 3.230. The 2018 I.M. 3.230 provides for shorter approach guardrail in locations with less than 400 vpd or 45-mph speed limit. However, "if ALL [emphasis is original] of the following conditions exist, the local public agency may elect not to install [approach] guardrail:

- Current ADT at structure is less than 400 vpd.
- Structure width (curb-to-curb) is 24 ft or greater, and is wider than the approach roadway width.
- Structure is on tangent alignment."(31)

I.M. 3.230 goes on to provide guidance for the installation of crash cushions or approach guardrail that is not considered crashworthy in some locations where the site conditions do not allow for the approach guardrail sections.

With respect to bridge rail on roadways with ADT of 400 vpd or less, I.M. 3.230 recommends that the Iowa DOT Bridge Standards be used to guide construction of bridge rail. If the designer decides not to use the Iowa DOT Bridge Standards on roads with less than 400 vpd then "a bridge rail considered to be crashworthy shall be used, meeting a minimum of TL-1 in NCHRP 350 or MASH."(31) Additionally, on short bridges, I.M. 3.230 allows designers to extend the guardrail using the Iowa DOT long span standard plan (BA-211) across the bridge rather than using a typical bridge rail, parapet, and approach section system.

Kansas DOT

In 2014, Kansas DOT performed a study to support guardrail and bridge rail recommendations for very low-volume local roads in Kansas. The bridges reviewed in this study were ones whose ADT was less than 50 vpd and when the bridge length was less than 50 ft. The study was a 5-year period from 2008 through 2012. Of the 306,056 crashes in Kansas during the study period, only 74 crashes occurred on the very low-volume roads that fit the inclusion criteria. Three of the crashes were coded as fatal crashes, there were 2 serious-injury crashes, and 69 crashes of other severities. Further, crashes on these short, VLV roads account for only 0.05 percent of the total fatal and serious-injury crashes and 0.0016 percent of all crashes in Kansas over the study period.(32) The authors go on to point out that:

> Due to the very small number of crashes with low-volume bridges the proportion of fatal crashes should be considered cautiously. Statistically the confidence interval for this small group is very large… As a result, one cannot interpret the data to indicate that a crash with a low-volume bridge is more likely to result in a fatality than any crash occurring in the State.(32)

Although there is support, both from the Kansas 2014 study and AASHTO guidance documents to eliminate the requirement for bridge rail on very low-volume roads with lengths of 20-50 ft an acknowledgement is made that some benefits of bridge rail cannot be captured by the study design. Examples of these benefits are that they provide delineation and even non-crashworthy barriers can provide some protection at low-angle and or low-speed crashes. Therefore, the final recommendation is that a non-tested bridge rail without approach guardrail design could be used on new or rehabilitated bridges that "meet all of the following conditions:

- The bridge is located on a road functionally classified as a Local Road.
- Traffic volume is less than or equal to 50 vpd.

- The approach roadway is a two-wheel path road.
- Roadway surface of approaches is gravel, sand, or dirt.
- Maximum length of the bridge is 50 ft.
- The new structure shall be no less than 24-ft wide.
- Bridge is not located on or adjacent to a curve or intersection.
- A Type 3 object marker shall be installed at each end of the bridge rails."[32]

In 2017 Kansas DOT performed a follow-up study to look at roadways with ADT over 50 vpd up to 100 vpd and bridges over 50-ft long up to 100-ft long. The study period was extended by two years to be from 2008 to 2014. Of the 424,494 crashes of all severity types that occurred over the study period 103 matched the inclusion criteria outlined above. Two of the crashes were categorized as fatal, 4 were serious-injury crashes, and 97 of the crashes were of other severities. The conclusion of the updated study is that:

> Although the crash risk is greater than the baseline risk found for bridges investigated in Report No. KS-14-16, the risk under any of these scenarios is very low. In addition, the costs of installing a crash tested bridge rail and properly installed approach guardrail section on these bridges cannot be justified because the return on the investment, measured in reduced crash costs, is very low.[33]

The updated recommendation for installation of non-tested bridge rails in the 2017 report are very similar to those from the 2014 study.[32, 33] The differences are highlighted below using **bold** type:

- "The bridge is located on a road functionally classified as a Local Road.
- Traffic volume is less than or equal to **100** vpd.
- The approach roadway is a two-wheel path road.
- Roadway surface on the approaches is gravel, sand, or dirt.
- Maximum length of the bridge is **100** ft.
- The new structure shall be no less than 24-ft wide.
- The bridge is not located on or adjacent to a curve or intersection.
- A Type 3 object marker shall be installed at each end of the bridge rails."[33]

Missouri DOT (MoDOT)

The MoDOT *Engineering and Policy Guide* covers guardrail, including bridge end treatments, in Section 606.1.[34] In this section, delineation is permitted in place of approach guardrail on bridges where the speed limit is less than 60 mph and the traffic volume is less than 400 vpd. The *Engineering and Policy Guide* goes on to recommend delineation-only on bridge replacements or rehabilitations where the existing structure was unshielded, or the existing roadway cannot reasonably accommodate the installation of approach guardrail. Further, the Guide dis-allows the delineation-only in the following situations: [34]

- On major roads and the National Highway System (NHS).
- For bridges in areas of poor geometry.
- In areas with crash history higher than the statewide average for similar roads.

South Dakota DOT

The South Dakota DOT *Local Roads Plan* published in 2011 includes design criteria for bridge rail on rural roads.[35] The plan allows turned down rail end treatments of 15 degrees or flatter and NCHRP 350 TL2 or better on rural collectors and local rural roads with ADT of less than 150 vpd.[35]

State Summary

Table 3 is a summary of the bridge rail and approach guardrail guidelines discussed in the previous sections. All of the States that the project team reviewed for this report have a traffic volume requirement to allow exceptions from typical bridge rail and approach guardrail design and placement guidance. Other conditions that States consider for exceptions include speed limit, structure width, and site layout. There is limited guidance for the installation of hardware on geometrically constrained sites.

Table 3. Summary of State practices for bridge rail and approach rail on low-volume roads.

State	Option	Condition
Illinois	Eliminate approach guardrail	ADT < 150; tangent; and/or bridge width ≥ roadway width or Owned by Twp; tangent; and/or bridge width ≥ roadway width.
Iowa	Shorter approach guardrail	ADT < 400; tangent; bridge width ≥ roadway width; and bridge width ≥ 24 ft.
	Crash cushion or non-tested approach guardrail	Above conditions and site layout does not allow shorter approach guardrail.
	Use TL-1 or better bridge rail or Long Span Standard Plan	ADT < 400.
Kansas	Non-tested bridge rail w/o approach railing	ADT < 100; Local road; 2-wheel path; gravel, sand or dirt approach; bridge length ≤ 100', bridge width ≥ 24 ft; not in proximity of curve or intersection; and use Type 3 object marker.
Missouri	Delineation in lieu of approach guardrail	ADT < 400; PSL < 60 mph; not on NHS or major road; areas of poor geometry; or high crash history.
South Dakota	Turn down rail end treatments and TL-2 or better bridge rail	ADT < 150; rural collectors and local rural roads.

Relevant Research

Bridge Curb and Railings for Extremely Low Volume Roads

Bigelow et. al performed a statistical analysis of low-volume bridges in Iowa and found that there were "fewer than 350 crashes over an 8-year period" that occurred on the over 17,000 low-volume bridges in the State.[36] Of those 350 crashes, 31 (10 percent) were fatal or serious-injury crashes. Bigelow et al.'s statistical analysis found that the crash rate increased with decreasing bridge width and decreasing traffic volume. The crash rate for bridges with fewer than 50 vpd was twice that of bridges with 200 to 400 vpd. Similarly, bridges less than 20-ft wide had a crash rate twice that of bridges that were between 24- and 28-ft wide. Bigelow et al. also found that a B/C analysis on a system-wide basis never resulted in a benefit-cost ratio greater than one where the alternative was to bring the railing "up to standard."

Seitz examined the issue of selecting bridge railing and approach guardrails for very low-volume roadways in Kansas.(32) Of the 3,600 deficient bridges in Kansas, 2,500 (70 percent) were bridges with traffic volumes less than 50 vpd. There were 74 crashes on bridges located on rural minor collectors or rural local roads in Kansas in the period 2008 through 2012. Almost seven percent of these crashes were fatal or serious injury. Like Bigelow et al., Seitz found that a B/C analysis where the alternative was updating to w-beam bridge rail was never greater than one on a systemwide basis.

Approach Guardrail for Bridges on Low-Volume Roads

Gates et. al performed a statistical analysis of:

> Ninety-six run-off-the-road crashes that occurred on the approach or departure to 68 county State-aid highway bridges in 10 Minnesota counties over a 15-year period.… None of the 33 crashes with approach guardrail resulted in a fatality or severe injury, whereas roughly one-quarter of the 63 crashes with a roadside bridge rail end resulted in a fatality or severe injury.(37)

The roadways in the analysis ranged from 16 to 41,524 vpd; with an average of 1,320 vpd and 325 vpd for bridges with and without approach railing, respectively. Additionally, 75 percent of the bridges were located on roads with a speed limit of 55 mph, 14 percent were located on roads with 40-50 mph speed limits and 11 percent were located on roads with speed limits between 30-35 mph.(37) Gates et. al concluded that the presence of approach guardrails lowers the severe crash rate except at bridges with the very lowest ADTs where crashes were infrequent.

Analysis Methods used in Roadside Design Guidance Development

Encroachment-based modeling has a rich, long history in roadside safety for the consideration and comparison of design alternatives. The 1977 *Barrier Guide* presented a hand-calculation method which AASHTO updated in 1989 when the first RDG was published.(25) Cost-effectiveness procedures which were based on the encroachment probability model were provided in Appendix A of the 1989 AASHTO RDG along with a computer program called *Roadside*. The Benefit-Cost Analysis Program (BCAP) followed the program *Roadside* with a focus on selecting bridge railings using a cost-benefit encroachment estimation procedure.(25)

Advancements to encroachment-based modeling and computing culminated in the Roadside Safety Analysis Program (RSAP) in 2003.(38) Field-collected encroachment trajectories and updates to crash severity modeling lead to a third update to RSAP in 2012, RSAPv3.(39) Most recently, advancements in statistical modeling have permitted the development of a simplified representation of encroachment trajectories leading to the simplification of encroachment-based computer modeling codes and ultimately a reduction in simulation time.(40) NCHRP Project 15-65, *Development of Safety Performance Based Guidelines for the Roadside Design Guide*, presents the current state-of-the-practice for encroachment-based modeling.(41)

The AASHTO RDG implies that the goal of roadside design is to reduce fatal and serious injury (KA) crashes. The objective of NCHRP Project 15-65 was to develop performance-based roadside safety guidance to address high-priority needs that support quantitative design decisions, and that promote consistency in interpretation and implementation.(41) NCHRP Project 15-65 developed a methodology which estimates the risk of fatal and serious injury crashes for use in evaluating roadside designs.(41)

The NCHRP Project 15-65 framework provides a governing equation to represent the sequence of run-off-the-road events and subevents to develop roadside designs which minimize the OUTCOME (e.g., risk, cost, etc.) of a crash as shown below in figure 1.(41)

$$\mathrm{OUTCOME_j} = \mathrm{ENCR_j}\ \mathrm{CRASH_j}\ \mathrm{SEV_j}$$

$$\mathrm{OUTCOME_j} = \left[\frac{\mathrm{BEF_S} \cdot \mathrm{EAF_S} \cdot \mathrm{L_S}}{5280}\right] \cdot \left[\mathrm{P_{c_j}} \cdot \prod_{i=1}^{j-1} \mathrm{THR_i}\right] \cdot \left[\mathrm{P_{SEV_j}} \cdot (1 - \mathrm{THR_j} \cdot \delta_j)\left(\frac{\mathrm{PSL_s^3}}{65^3}\right)\right]$$

$$\mathrm{OUTCOME_S} = \sum_{j=1}^{N} \mathrm{OUTCOME_j}$$

Figure 1. Equation. NCHRP 15-65 governing equation.

Where:

- $OUTCOME_S$ = The total number of crashes with the specified outcome on the segment involving all features on the segment.
- $OUTCOME_j$ = The number of crashes with the specified outcome involving feature j (e.g., the number of serious injury or fatal crashes involving impacts with a bridge curb) per edge mile per year.
- j = Feature number from 1 to n where n is the total number of features evaluated on the segment.
- BEF_S = The expected annual number of encroachments expected on a segment in edge encroachments/mi/yr assuming base conditions as a function of traffic volume (AADT).
- EAF_S = Highway and traffic characteristic encroachment adjustment factors for the highway segment of interest.
- L_S = Segment length in feet.
- P_{cj} = The conditional probability of a vehicle striking an object given an encroachment occurs. The length ratios are the probability of leaving the roadway in the given proportion of the roadway under the assumption that encroachments are equally likely anywhere on the segment. The form of P_{cj} depends on the type of object as shown below:

  Continuous Features (e.g., guardrails, curbs, edge of bridge, etc.) shown in figure 2.

$$P_{cj} = \left[\frac{L_j}{L_S}\right] \cdot P_{y(W_{Fj})}$$

Figure 2. Equation. Continuous features.

Discreet Features (e.g., trees, poles, bridge piers, water bodies, etc.) shown in figure 3.

$$P_{cj} = \left[\frac{L_j}{L_S}\right] \cdot P_{y(W_{Fj})} + \left[\frac{L_{TMax}}{L_S}\right]\left[P_x(L_{TMax})(P_{y(W_{Fj})} - P_{y(W_{Bj})})\right]$$

Figure 3. Equation. Direct features.

- P_{SEVj} = The conditional probability of observing the severity of interest given that there is an interaction with roadside feature j.
- THR_j = The conditional probability of passing through feature j given the vehicle interacts with feature j.
- δ_j = One if all interactions with the feature do not lead to an increase in harm (e.g., terrain); zero if all interactions with the feature lead to an increase in harm (e.g., longitudinal barrier, edge of bridge).
- PSL_s = Posted speed limit on the segment in mi/hr.
- L_j = The effective length of an individual feature j along the segment in ft.
- $P_y(Y_j)$ = Cumulative probability density function of the lateral extent of encroachment when lateral offset y = Y.
- $P_x(X_j)$ = Sum of the cumulative probability density function of the maximum longitudinal extent of encroachment.
- W_{Bj} = The distance in feet from the edge of the traveled way measured laterally to the farthest point of feature j plus $V_W \cos(\theta_{15})$.
- W_{Fj} = The distance in feet from the edge of the traveled way to the closest face (i.e., traffic side) of feature j.
- L_{TMax} = The length in ft of the longest trajectory in the data base of trajectories used to calculate $P_x(X_j)$ and $P_y(Y_j)$ (i.e., 1,000 ft).
- V_w = Typical passenger vehicle width in ft (e.g., 6.5 ft).
- θ_{15} = The 15^{th}-percentile encroachment angle in degrees (e.g., 5 degrees[42]).
- θ_{85} = The 85^{th}-percentile encroachment angle in degrees (e.g., 22 degrees[42]).

More details on the derivation of figure 1 can be found in the NCHRP Project 15-65 final report.[41] Recalling the goal of roadside design involves minimizing the frequency of KA run-off-road crashes, there are a variety of approaches that can be used after the frequency of KA crashes for each alternative has been determined using figure 1.

Risk is the proportion of poor outcomes to all outcomes. Relative risk can be used to determine if the risk of the null alternative is greater or less than the risk of the alternatives under consideration.[43-45] The null alternative is the existing conditions of the roadside being evaluated.

The B/C method used by programs like Roadside, BCAP, RSAP, and RSAPv3 estimate the societal cost of crashes through estimating the risk of crashes for the modeled alternatives. The societal costs of crashes are then compared to the agency costs such as construction, maintenance, and repair for each alternative over the life of the project. The benefit is usually considered to be the reduction in societal costs associated with each alternative compared to the direct costs (i.e., the construction, maintenance, and repair costs). A B/C ratio greater than one indicates the alternative should be considered.

Cost effectiveness analysis is similar to B/C analysis but instead of monetizing the societal cost of the crash reduction (i.e., the benefit), the number of fatal and serious injury crashes avoided is used. An incremental cost effectiveness ratio (ICER) compares the annualized direct costs of the design alternative (i.e., construction, maintenance, repair) to the annual reduction in the number of KA crashes. The ICER is defined as follows in figure 4:

$$\text{ICERi/j} = \frac{DC_i - DC_j}{PO_j - PO_i}$$

Figure 4. Equation. ICER.

Where:

- ICERi/j = Incremental cost effectiveness ratio of alternative j with respect to alternative i.
- PO_i, PO_j = Performance outcome for alternatives i and j over the project life.
- DC_i , DC_j = Annualized direct costs for alternatives i and j.

The ICER can be thought of as the annual cost of the design alternative required to avoid one KA crash. For example, one bridge rail alternative may have an annualized cost of construction, maintenance, and repair of $5,000 to avoid one KA crash per year. Another alternative may have an annualized cost of $10,000. The alternative with the lower ICER would be the preferred alternative since it accomplishes the objective of avoiding one KA crash at a lower cost. The selected alternative may or may not be cost-beneficial but choosing the lowest ICER promotes the best use of scarce agency funds. The ICER could be calculated for various bridges and used to prioritize improvements to those bridges.

Forest Service Manual FSM 7700

Forest Service requirements for the design and construction of bridges are included in Chapter 7720 (Transportation System Development) of the *Forest Service Manual 7700* (Travel Management) and Chapter 72 (Design Requirements) of FHS 7709.56b.(46, 47) FSM 7722.03 indicates that all bridge designs be consistent with the AASHTO LRFD bridge design specifications.(48) Requirements for traffic barriers and bridge railings are discussed in FSM 7722.12 and FSH 7709.56b Section 72.3.

FSM 7722.12 (Traffic Barriers and Approach Guardrails) indicates that the AASHTO guidelines for VLV roads and RDG be used. (24, 26) Further, Section 7722.12 states that "all road bridges must be designed with a traffic barrier system that has either been successfully crash-tested to a currently acceptable TL per the 2009 AASHTO MASH." (14) Bridges are categorized into maintenance levels 1 through 6 which are then associated with MASH test levels as shown in table 4. Approach guardrails are not required for the TL1 systems but must be used for TL2 and TL-3 if the roadway approach geometry allows.

Table 4. Association of maintenance levels and MASH test levels.[47]

Maintenance Level (ML)	Minimum Required Test Level (TL)
MLs 1 and 2	TL 1
ML 3 with design speed ≤ 30 miles per hour	TL 1
ML 3 with design speed > 30 miles per hour	TL 2
MLs 4 and 5	TL 3

Section 72.3 of FSH 7709.56b (Roads Bridge Traffic Barriers) also discusses bridge railings. Like FSM 7722.12, it references the AASHTO LRFD bridge design specifications, the VLV Guide, RDG, and MASH.[14, 24, 26, 41, 49] Crash tested bridge railings are required and the test level and maintenance level associates shown in table 4 are reiterated. Approach guardrail for bridges is discussed in Section 72.4 (Approach Guardrail for Road Bridges).

As discussed above, the Forest Service requirements explicitly cite the relevant AASHTO documents.

Manual on Uniform Traffic Control Devices

Chapter 5 of the *Manual on Uniform Traffic Control Devices* (MUTCD) addresses traffic control devices for low-volume roads. [50] Chapter 5 provides guidance for bridge signage for narrow bridges, one-lane bridges and object markers, and barricades for the ends of roads. [50]

When objects are located adjacent to the roadway, such as bridge abutments or the ends of traffic barriers, the MUTCD requires that a Type 3 marker be used. Type 3 markers are "vertical rectangles with alternating black and retroreflective yellow stripes sloping downward at an angle of 45 degrees towards the side of the obstruction on which traffic is to pass."[50] The MUTCD provides the following guidance for object markers mounted on posts: when marking objects within the roadway or closer than 8 ft from the shoulder, the minimum mounting height from the surface of the road way to the bottom of the object marker should be 4 ft. It goes on to provide the option that "when object markers are applied to an obstruction that by its nature requires a lower or higher mounting, the vertical mounting height may vary according to need."[50]

Type 3 objects markers are applied to roadside appurtenances. Type 3 object markers may be mounted directly to a bridge abutment or rail end or mounted on a signpost. If mounted directly on the appurtenance the marker height will be dictated by the geometry of the appurtenance, if mounted on a sign post the height to the bottom of the object marker should be 4 ft.

Summary

The proceeding review of literature and crash data studies compiles the existing guidance for bridge rail design on low- and VLV roads. There is some variety in the ways each State addresses low-volume roads. Guidance for extremely low volumes (i.e., less than 50 vpd) and low-speed roads is limited. One observation that is repeated throughout this literature review, however, is that upgrading bridge rails on bridges servicing low-volume roads is generally not cost-beneficial. Conversely, however, there are crash risks associated with these bridges which must be balanced with the costs of improvements to these bridges.

Available, crash tested hardware appropriate for use in low-speed situations for containing passenger vehicles was also gathered. While not overwhelming in scope, there is an assortment of rails, transitions, and terminals available for these low-speed situations. The Guide includes new construction drawings of these available solutions.

PARAMETERS FOR USE OF BRIDGE RAILS AND ASSOCIATED HARDWARE

After developing the structure and layout for the Guide, the project team made recommendations for the various parameters of the Guide. This section documents the development of those parameters.

Establishing Safety Performance Goals

A safety performance goal measures the reduction in risk to passenger vehicle occupants resulting from installing or improving a roadside feature. Roadside hardware should only be installed if it results in a reduced risk of fatal or serious passenger vehicle crashes (KA).

Shielding a shallow drainage ditch with several hundred feet of guardrail and terminals, for example, may actually increase the risk since the guardrail is more likely to be struck because it is much longer than the culvert. On the other hand, installing a bridge railing to prevent vehicles from leaving the bridge and falling from a large height into a water-filled channel likely will reduce the risk of a fatal or serious injury crash. A safety performance goal (PG) is the amount of risk reduction necessary to justify implementing a proposed improvement.

The project team considered relative risk reduction and benefit cost PGs. Given the previously observed challenges with using a B/C approach, the project team recommended a relative risk reduction approach.

Bridge Railings

This project was concerned with evaluating bridge railings on extremely low-volume, low-speed roadways. In particular, three speed categories were considered (i.e., 5-15 mph, 16-30 mph, and 31-45 mph) and all the bridges experienced a traffic volume of 50 vpd or less. Ray et al. recently developed a risk-based method for examining roadside designs.(51) The method predicts the number of expected fatal and serious-injury crashes for a roadside design based on statistical models that incorporated the traffic characteristics and placement of roadside features. The expected number of fatal and serious injury crashes are called the OUTCOME of the design. If a proposed roadside design results in fewer fatal and serious-injury crashes than the existing design, the following equation will hold true (as shown in figure 5):

$$1 > \frac{\mathrm{OUTCOME}_{\mathrm{PROPOSED}}}{\mathrm{OUTCOME}_{\mathrm{EXIS\ TING}}} = \frac{\mathrm{ENCR}_{\mathrm{PROPOSED}}\ \mathrm{CRASH}_{\mathrm{PROPOSED}}\ \mathrm{SEVERITY}_{\mathrm{PROPOSED}}}{\mathrm{ENCR}_{\mathrm{EXISTING}}\ \mathrm{CRASH}_{\mathrm{EXISTING}}\ \mathrm{SEVERITY}_{\mathrm{EXISTING}}}$$

Figure 5. Equation. Risk of proposed to existing crashes.

where

OUTCOME	=	The expected annual frequency of fatal and serious-injury crashes for the roadside design.
ENCR	=	The expected number of vehicle encroachments onto one roadside edge.
CRASH	=	The conditional probability of a crash with a roadside feature given an encroachment occurs.
SEVERITY	=	The conditional probability of a fatal or serious injury given that a crash occurs.

Each of the three terms (i.e., ENCR, CRASH, and SEVERITY) are estimated by detailed mathematical and statistical models presented in the NCHRP 15-65 final report. [51] In the case of evaluating the need for a bridge railing, the ENCR and CRASH terms will be identical for both the existing design (e.g., no bridge railing) and the proposed design (e.g., install a crash tested a bridge railing). The ENCR term involves characteristics like the traffic volume, posted speed limit, horizontal and vertical curvature, and other similar roadway characteristics. Since none of these are changed in assessing the need for a bridge railing the $ENCR_{PROPOSED}$ will be equal to $ENCR_{EXISTING}$. Similarly, the CRASH term involves the location and type of roadside features. Since the bridge railing is located at the edge of the bridge deck both $CRASH_{PROPOSED}$ and $CRASH_{EXISTING}$ will be equal. Evaluating the need for bridge railing, then, is only a function of the ratio of the $SEVERITY_{PROPOSED}$ to $SEVERITY_{EXISTING}$. The risk reduction (RR) ratio of OUTCOMEs for evaluating bridge railings is simply (as shown in figure 6):

$$\frac{\text{SEVERITY}_{\text{BRIDGE RAILING}}}{\text{SEVERITY}_{\text{NO BRIDGE RAILING}}} = \text{RR}_{\text{BR}|\text{NO BR}}$$

Figure 6. Equation. Relative risk of bridge rail to no bridge rail.

This ratio for the case of evaluating bridge railings is the relative risk and should be less than unity to consider installing a bridge railing. Another related term is the relative risk reduction (RRR) which is given by figure 7:

$$\text{PG} = \text{RRR}_{\text{BR}|\text{NO BR}} = \frac{\text{SEVERITY}_{\text{NO BRIDGE RAILING}} - \text{SEVERITY}_{\text{BRIDGE RAILING}}}{\text{SEVERITY}_{\text{NO BRIDGE RAILING}}}$$

Figure 7. Equation. Relative risk reduction of bridge rail to no bridge rail.

The PG is the particular value of RRR needed to justify the construction of a bridge railing. Calculating the RRR for evaluating bridge railing need involves calculating the crash severity of the following:

- A vehicle falling from an unshielded bridge deck onto a dry or water-covered surface below the bridge from various heights.
- A vehicle striking and penetrating through a bridge railing onto a dry or water-covered surface below the bridge from various heights.
- A vehicle striking a crash tested bridge railing.

In addition, the following simplifying assumptions were made in the analysis:

- A bridge railing that has not been evaluated in full-scale crash tests is assumed to be effective in containing passenger vehicle crashes half the time.
- A crash tested bridge railing will always contain and redirect a passenger vehicle (i.e., the chance of penetrating a crash tested bridge railing is so small that it may be neglected).

It is recognized that neither of these assumptions will be true under all impact conditions, however, the assumptions were necessary due to the lack of data to quantify either effectively.

SEVERITY is expressed as the proportion of fatal and serious injury crashes (KA) with respect to all crashes involving the feature. The proportion of fatal and serious crashes (KA) are adjusted to a baseline speed of 65 mph based on a method proposed by Ray and Carrigan as shown in figure 8: [52]

$$\mathrm{SEVERITY_j} = \mathrm{KA_{65}}\left[\frac{\mathrm{PSL}^3}{65^3}\right]$$

Figure 8. Equation. The condition probability of a fatal or serious injury given that a crash occurs.

where

KA	=	The proportion of fatal and serious injury crashes involving feature j with respect to all crashes with feature j adjusted to a baseline speed of 65 mph as per. [52]
PSL	=	Posted speed limit in mph.

Carrigan found that the proportion of fatal and serious-injury crashes for rigid concrete bridge rails and median barriers was 0.0159 normalized to a posted speed limit of 65 mph. [53] There are no in-service evaluations of test TL-1 or TL-2 bridge railings, so this value is used as a estimate for bridge railings on low-speed, low-volume roadways. Similarly, Ray found that the proportion of fatal and serious injury crashes when vehicles were immersed in water was 0.0486 based on the 2002 to 2006 Washington State crash data. [39] In a study examining selection criteria for highway bridges, Ray proposed a risk of serious and fatal injury crashes of 0.0589 for environments under a bridge where only the vehicle occupants were at risk.[17]

These values are shown in the bottom row of table 5. The severity values for bridge rail crashes (KA_{BR}) are scaled by the highest posted speed limit in the three speed limit categories as shown in the second column of table 5. It has been assumed that immersion in water or a drop of 20 ft or more is not a function of posted speed so all the severity values for water surfaces in the fourth and last columns of table 5 are the same. The severity value for a dry surface condition under the bridge for drops of 20 ft or less and a posted speed limit of 31-45 mph was found by scaling the low-risk environment value by the posted speed.

Under bridge conditions with 6 or more inches of water have a higher risk than dry under bridge conditions when the drop is 20 ft or less because trapped vehicle occupants have been known to drown in water channels with as little as 6 inches of water.

Table 5. Crash severity proportions for bridge features at various posted speed limits.

Posted Speed Limit (mph)	**KA_{BR}**	**KA_{DRY}**			**KA_{WATER}**		
		Drop Height - H (ft)					
		H ≤ 20	**20 < H ≤ 50**	**H > 50**	**H ≤ 20**	**20 < H ≤ 50**	**H > 50**
5 - 15	0.0002	0.0002	*0.2501*	0.5000	0.0486	*0.2743*	0.5000
16 - 30	0.0016	*0.0099*	*0.2549*	0.5000	0.0486	*0.2743*	0.5000
31 - 45	0.0053	0.0195	*0.2598*	0.5000	0.0486	*0.2743*	0.5000
65	0.0159	0.0589	-	-	0.0486	-	-

There are no data available that can be used to estimate the crash severity of passenger vehicles falling from a bridge deck onto a surface. Weckbach has summarized recent medical research on survivability in free falls from high drops as follows:

> "The American College of Surgeons' Committee on Trauma (ACS-COT) defines a critical threshold for a fall height in adults as > 20 feet (6 meters), as part of the field triage decision scheme for transport to a designated trauma center. A retrospective analysis of 101 patients who survived vertical deceleration injuries revealed an average fall height of 23 feet and 7 inches (7.2 meters), confirming the notion that survivable injuries occur below the critical threshold of a falling height around 20-25 feet. A more recent study on 287 vertical fall victims revealed that falls from height of 8 stories (i.e., around 90-100 feet) and higher, are associated with a 100% mortality. Thus, a vertical falling height of more than 100 feet is generally considered to constitute a "non-survivable" injury". (54)

Turgut et al. also summarized the survivability of free falls from heights finding similar results. (55) Interestingly, the velocity of a free fall from 20 ft is $\sqrt{2(32.2)20} = 35.9$ ft/s which is just under the 40 ft/s maximum allowable occupant impact velocity used in crash testing guidelines like Report 350 and MASH. (12, 15) These results indicate that a person may survive jumping from a bridge with a 20-ft drop although there will likely be injuries. If a 20-ft drop is survivable for a person jumping off a height, a vehicle occupant should be no more at risk since they have the added benefit of passive safety systems (e.g., front and side airbags, seat belts, roof-crush protection, etc.). A vehicle dropping off a bridge 20 ft or less above a dry surface at a low posted speed limit (i.e., 5-15 mph) would likely not result in a fatal or serious injury based on the summaries of the biomedical literature presented by Weckbach and Turgut.(54)

Lapostolle et al. also examined factors affecting survivability of falls from heights and found that the median height for a fatal fall was 49 ft and virtually all victims are fatally injured in a fall of 85 ft or more. (56) Based on the median height for fatal injuries from Lapostolle et al., severity of passenger vehicles falling off bridges with drops of 50 ft or higher over either wet or dry surfaces was taken to be 0.5000 as shown in table 5. Ray examined bridge rail crash data from

Pennsylvania, Ohio, and Nebraska and found 38 bridge rail penetrations.(17) The 38 bridge-rail penetrations involved:

- 0 motorcycles.
- 26 passenger cars.
- 12 heavy vehicles.

The 38 bridge-rail penetrations resulted in 5 fatal crashes, 13 A-injury crashes, 5 B-injury crashes, 6 C-injury crashes, 8 PDO crashes, and 1 crash of unknown severity. This resulted in crash severity of 0.4737 (i.e., (5+13)/38=0.4737). This value found by Ray (0.4737) confirms the recommendation of Lapostolle et al. from the biomechanics literature. Independently, the values agree quite well. Values shown in an italic font in table 5 are interpolated from the values to the left and right. Given these crash severity estimates, risk reduction ratios can be calculated as shown in table 6.

Table 6 shows the relative risk reductions for bridge railings on low-speed, low-volume bridges. If the PG is that the RRR must be greater than 0, a bridge railing meets the PG for all conditions except bridges with a 20 ft or smaller drop to a dry under bridge environment.

Table 6. RRR for bridge railings on bridges with posted speed limits of 45 mph or less and 50 vpd or less.

Posted Speed Limit (mi/hr)	**Dry Environment Under Bridge** (Condition A)			**Water Under Bridge** (Condition B)		
	Drop Height - H (ft)					
	H ≤ 20	**20 < H ≤ 50**	**H > 50**	**H ≤ 20**	**20 < H ≤ 50**	**H > 50**
5 - 15	0%	100%	100%	88%	100%	100%
16 - 30	98%	99%	100%	88%	99%	100%
31 - 45	97%	98%	99%	88%	98%	99%

A = An ephemeral or intermittent stream channel with water depth less than six inches during the majority of the time the bridge is in use.
B = Any stream channel with year-round flow or ephemeral or intermittent stream channels with water depths greater than or equal to six inches for at least three months.

Virtually all the situations studied indicate a bridge rail would satisfy the PG, except for heights equal to 20 ft or less, over a drive environment at posted speed limits of 5-15 mph. Using the posted speed limit grouping and the impact speeds used when developing the various Test levels, the selection guidelines shown in table 7 indicate which test level of rail is appropriate for which speed.

Table 7. TL selection for bridge rails on roads with ADT ≤ 50 vpd and posted speed ≤ 45 mph.

Posted Speed Limit (mph)	**Dry Environment Under Bridge** (Condition A)			**Water Under Bridge** (Condition B)		
	Drop Height - H (ft)					
	H ≤ 20	**20 < H ≤ 50**	**H > 50**	**H ≤ 20**	**20 < H ≤ 50**	**H > 50**
5 - 15	NRB	12-in curb rail or TL-1				
16 - 30	TL-1 or higher bridge rail					
31 - 45	TL-2 or higher bridge rail					

Note: Surface condition under bridge:
A = An ephemeral or intermittent stream channel with water depth less than six inches during the majority of the time the bridge is in use.
B = Any stream channel with year-round flow or ephemeral or intermittent stream channels with water depths greater than or equal to six inches for at least three months.
NRB =Not risk beneficial.

Approach Terminals and Transitions

Establishing a, PG for the terminals and transitions on the approach to an extremely low-volume, low-speed bridge follows a similar path to a bridge railing PG.

There are only a few MASH TL-1 and TL-2 guardrail terminals and there are no in-service performance evaluations (ISPEs) to determine the appropriate crash severity. Crash severity of Report 350 TL-3 terminals has been examined in a few studies. Ray found that less than five percent of crashes with Report 350 tangent guardrail terminals resulted in fatal or serious injuries.(57) Similarly, Carrigan and Ray found that a little over five percent of crashes with Report 350 TL-3 flared guardrail terminals resulted in fatal or serious injuries.(58) A 65-mph baseline crash severity of 0.0500 appears to be a reasonable assumption for TL-1 and 2 terminals based on these studies.

The National Highway Traffic and Safety Administration (NHTSA) requires passenger vehicles meet the requirements of the Federal Motor Vehicle Safety Standards (FMVSS).(59) Several of the FMVSS requirements are crash tests. FMVSS 208 requires that passenger vehicles pass safety requirements in a 35-mph full frontal crash into a flat rigid wall. Since the vehicle safety systems are designed to protect vehicle occupants in collision with rigid objects up to an impact speed of 35 mph no terminal is necessary to shield the bridge rail end in the lower two speed categories (i.e., posted speed limits 30 mph or less).

There are no ISPEs of transitions but the severity of strong-post w-beam guardrail normalized to 65 mph has been found to be 0.0071.(60) It is assumed that crash tested transitions have risk values similar to strong-post w-beam guardrails. One of the most rigid roadside features on the roadway are bridge piers. Ray found that the severity of bridge pier crashes normalized to 65 mph is 0.0656.(49) The risk value for a rigid pier is used to conservatively represent striking the end of a bridge railing. These 65-mph baseline risk values are shown in the bottom row of table 8.

Table 8. Crash severity proportions for terminals, transitions, and unshielded bridge rail ends at various posted speed limits.

Posted Speed Limit (mi/hr)	$KA_{EXPOSED\ END}$	KA_{TERM}	KA_{TRANS}
5 - 15	0.0000	0.0006	0.0000
16 - 30	0.0000	0.0049	0.0000
31 - 45	0.0218	0.0166	0.0024
65	0.0656	0.0500	0.0071

As was done earlier for bridge railings, the relative risk reduction can be calculated from the values in table 8 as shown below in table 9 where the indication "NRB" indicates that a terminal or transition is not risk beneficial for the conditions.

If the posted speed limit is 30 mph or less and the rail is terminated at a post when a post and rail bridge rail system is used, the installation of new terminals and/or transitions is generally not risk beneficial. If the posted speed limit is greater than 30 mph, the installation of terminals and transitions are risk beneficial.

Table 9. RRR for terminals, transitions, and unshielded bridge ends on bridges with posted speed limits of 45 mph or less and 50 vpd or less.

Posted Speed Limit (mi/hr)	**Terminals**	**Transition**
5 - 15	NRB	NRB
16 - 30	NRB	NRB
31 - 45	24%	89%

NRB = Not risk beneficial.

NEW HARDWARE DEVELOPED

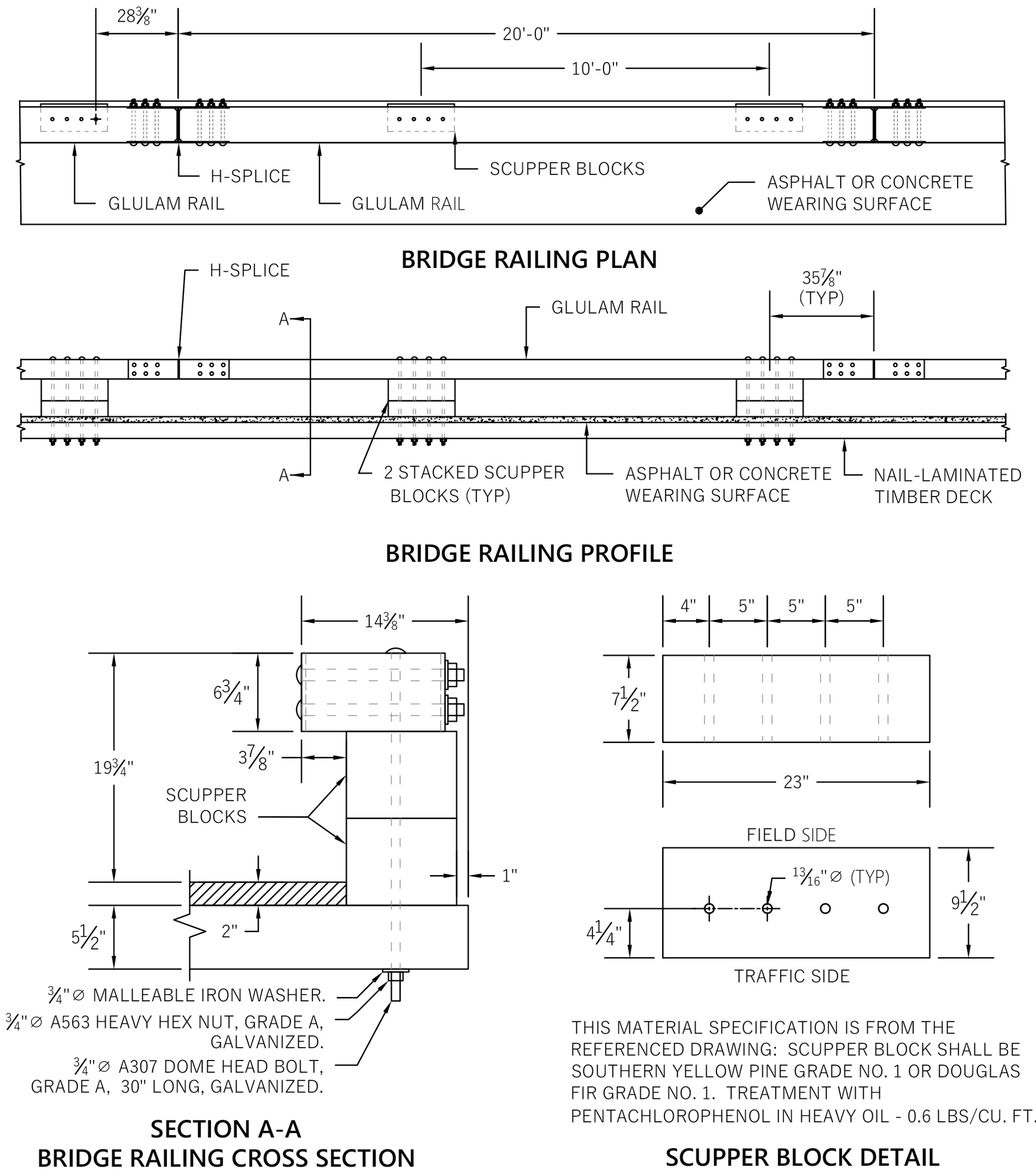
One of the objectives of this project was to modify the MASH TL-1 low profile WVBR for installation on a lower strength bridge structure (e.g., 4x12 wood plank bridge deck), and then assess crash performance under MASH TL-1 conditions using finite element analysis (FEA).[15] A summary of that effort is provided below.

The WVBR is a 19.75-inch tall timber bridge rail that was developed and full-scale crash tested to MASH TL-1 by MwRSF in 2008.[61] Figure 9 shows an engineering drawing for the MASH TL-1 WVBR bridge rail (additional drawing details can be found in the Guide). The full-scale test involved the WVBR bridge rail mounted onto a transverse, nail-laminated timber deck constructed from 2-by-6 pine boards. The impact conditions included a 2002 Dodge Ram 1500 Quad Cab pickup with a gross static mass of 5,179-lb impacting the railing at 30.8 mph and impact angle of 26.1 deg.

A detailed FEA model of the baseline WVBR bridge rail including a portion of the bridge superstructure and deck was developed by the project team, as illustrated in figure 10. The project team then validated the FEA model against a full-scale crash test WVBR-1 using the procedures outlined in NCHRP Web-Document 179. Once the model was validated, the mount design was modified to accommodate installation of the WVBR bridge rail on lower strength bridge decks.

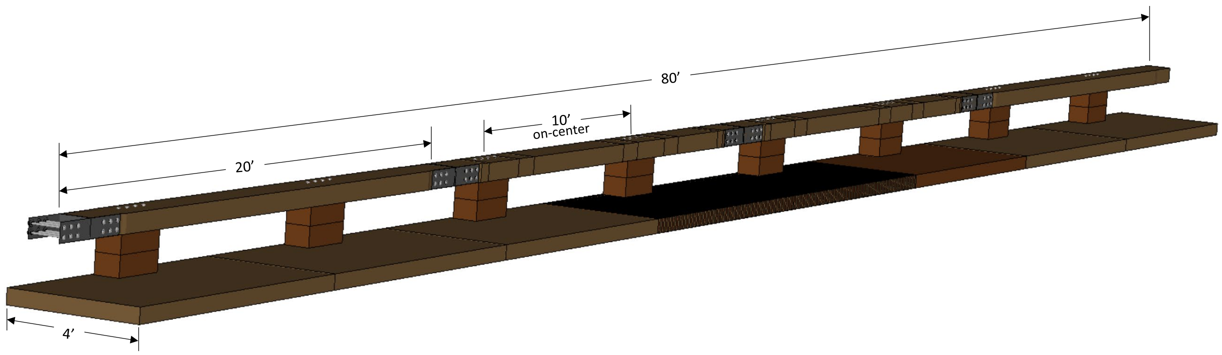
The project team communicated with the project's technical advisory committee (TAC) to identify the critical bridge deck design. The TAC included representatives from the Bureau of Reclamation, the Bureau of Indian Affairs, Bureau of Land Management, and National Parks Service (Great Smoky Mountain Park). Each of these organizations have an interest in bridge rails used on extremely low-traffic, low-speed timber bridges. The project team found that the 2-by-4 laminated deck was the most common deck type among the user agencies, but that the 4-by-12 plank deck was more critical for bridge rail mounting. Both decks are 3.5 inches thick.

The project team then developed a finite element model of a portion of a bridge superstructure design including a 4-by-12 timber plank deck. The mount for the baseline WVBR was modified to meet connection strength requirements for the lower strength deck design. The scope of this effort was limited to developing a modified mount; thus, the baseline bridge rail design (i.e., rail, scupper blocks and hardware) was not modified. It was determined from review of damages to the bridge deck during the static testing that the overall strength of the post-mount connection was likely reduced due to the malleable washers crushing into to deck boards during loading. Thus, a primary aspect of the modified design was to include a load distributor plate on the bottom surface of the deck to spread the forces over a larger area to prevent these local damages.



Source: FHWA

Figure 9. Graphic. Engineering drawing of the MASH TL-1 WVBR bridge rail.



Source: FHWA

Figure 10. Graphic. FEA model of the MASH TL-1 WVBR bridge rail test article (oblique view).

Three alternative mount designs were developed and are illustrated in figures 11, 12, and 13. Each or these designs were shown to meet all performance criteria in MASH for TL-1 impact conditions. The barrier successfully contained and redirected the pickup with minimal damage to the system. There were no detached elements from the barrier that showed potential for penetrating the occupant compartment or presenting undue hazard to other traffic. The vehicle remained upright and very stable throughout impact and redirection. The occupant impact velocity (OIV) and maximum occupant ridedown accelerations (ORA) values were within recommended limits specified in MASH. Table 10 shows the results for the occupant risk calculations and includes additional metrics specified by the European Committee for Standardization. MASH evaluation criteria recommends that OIVs be less than 30 ft/s, and that ORAs be less than 15 g.

Accordingly, the WVBR bridge rail with the each of the modified mount designs is considered applicable to other bridge structures with similar or greater deck-strength at the bridge rail mounting points (e.g., thicker wood decks, stronger materials, etc.). Design 3, which is shown in figure 13, provides the greatest strength and is the recommended mounting option for bridges with transverse plank decks.

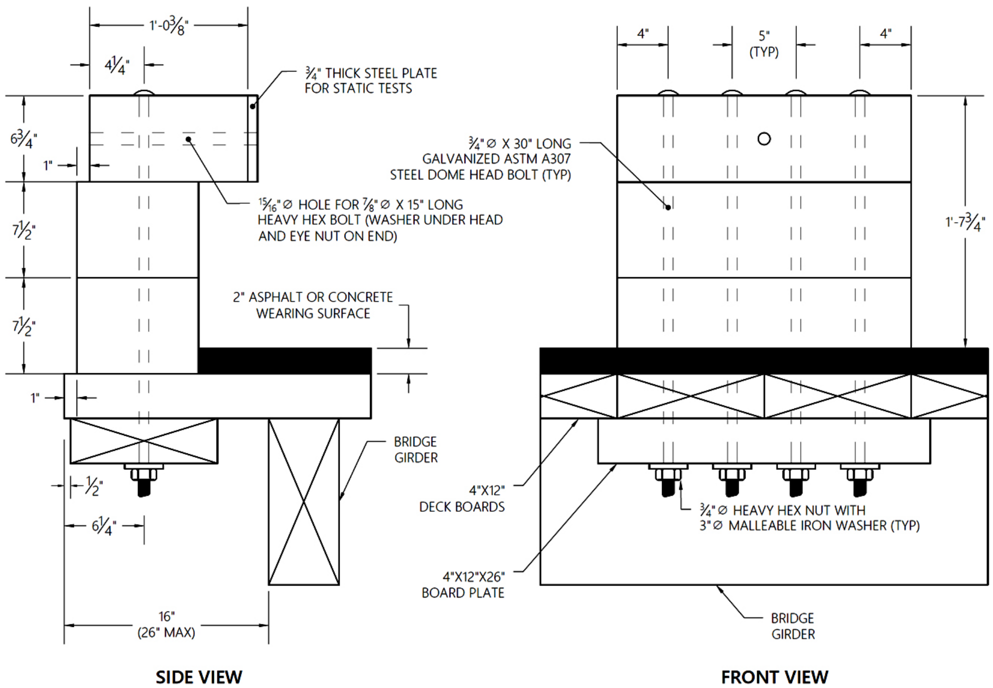


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Figure 11. Graphic. Design alternative 1 for modified WVBR for use on 4-by-12 plank decks.

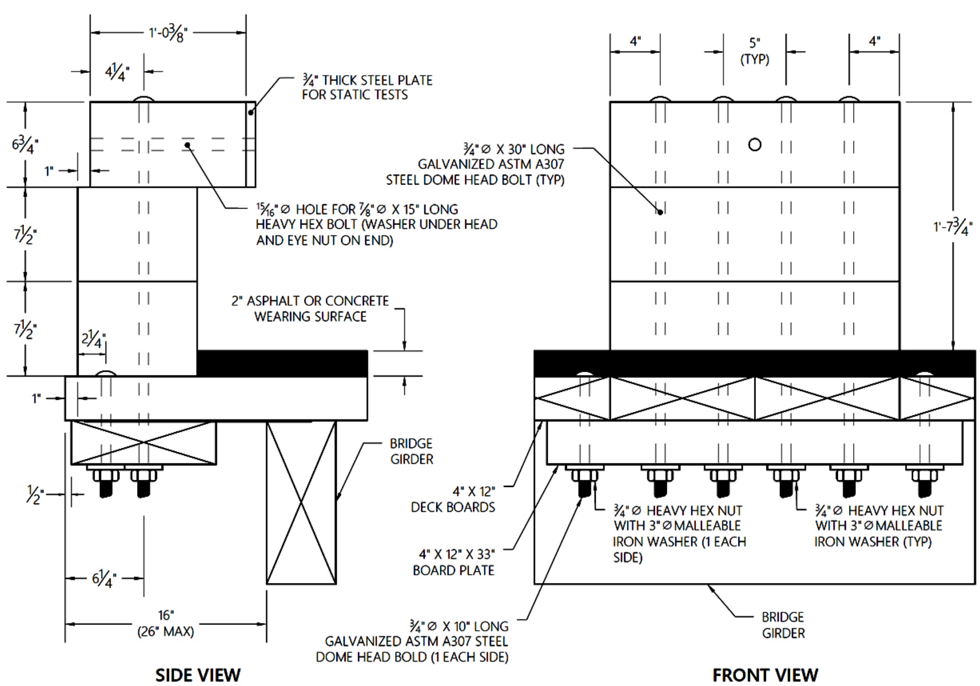

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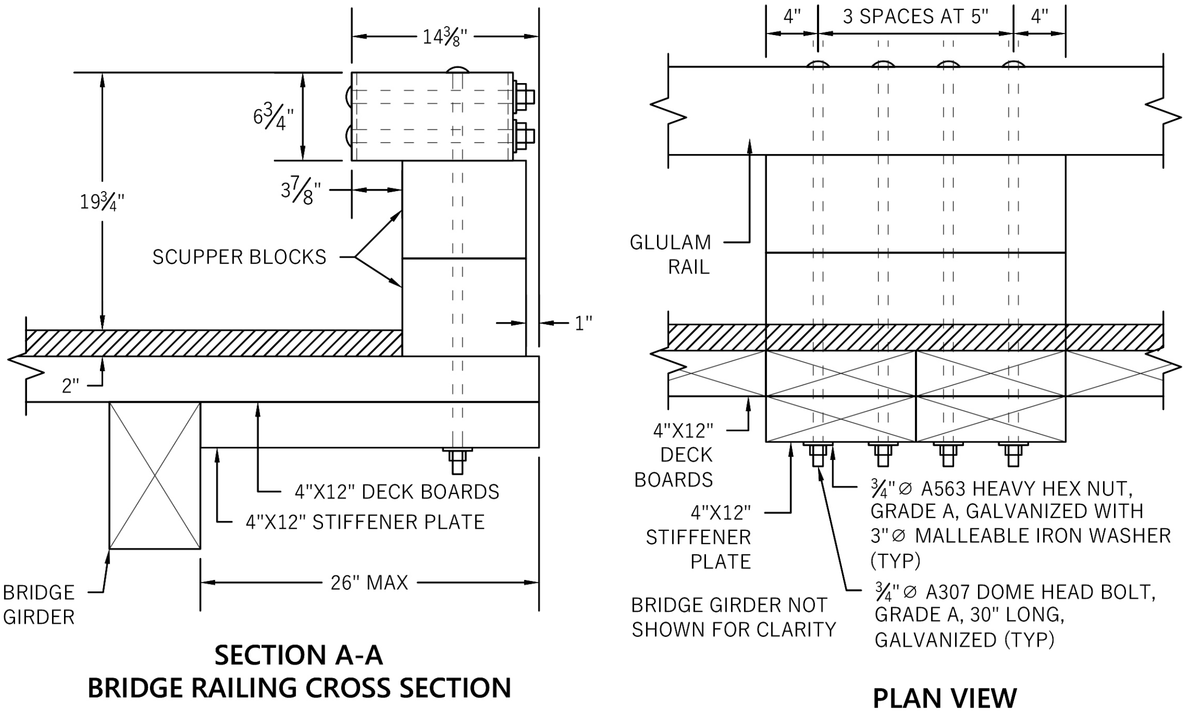
Figure 12. Graphic. Design alternative 2 for modified WVBR for use on 4-by-12 plank decks.



Source: FHWA

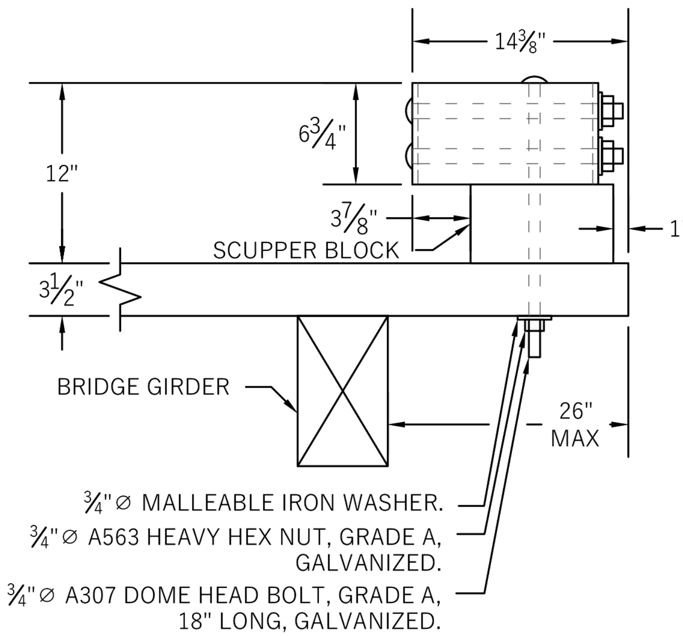
Figure 13. Graphic. Design alternative 3 (recommended) for modified WVBR design for use on 4-by-12 plank decks.

Table 10. Occupant risk metrics computed using TRAP software for the baseline and modified mount cases.

Occupant Risk Factors		MASH 1-11			
		Baseline	D1	D2	D3 (26" OH)
Occupant Impact Velocity (ft/s)	x-direction	11.8	11.5	11.8	11.5
	y-direction	13.8	13.5	13.5	13.1
	at time	at 0.1609 seconds on right side of interior	at 0.1638 seconds on right side of interior	at 0.1639 seconds on right side of interior	at 0.1550 seconds on right side of interior
THIV (ft/s)		18.0 at 0.1554 seconds on right side of interior	17.1 at 0.1579 seconds on right side of interior	17.4 at 0.1581 seconds on right side of interior	17.7 at 0.1550 seconds on right side of interior
Ridedown Acceleration (g's)	x-direction	-5.4 (0.5422 - 0.5522 seconds)	-5.9 (0.1865 - 0.1965 seconds)	-5.8 (0.2224 - 0.2324 seconds)	-5.2 (0.5535 - 0.5635 seconds)
	y-direction	-3.3 (0.6418 - 0.6518 seconds)	-3.2 (0.2269 - 0.2369 seconds)	-3.9 (0.2004 - 0.2104 seconds)	-3 (0.5496 - 0.5596 seconds)
PHD (g's)		5.7 (0.5422 - 0.5522 seconds)	6 (0.1866 - 0.1966 seconds)	6.2 (0.2226 - 0.2326 seconds)	5.7 (0.5495 - 0.5595 seconds)
ASI		0.46 (0.0508 - 0.1008 seconds)	0.45 (0.0507 - 0.1007 seconds)	0.45 (0.0502 - 0.1002 seconds)	0.47 (0.0517 - 0.1017 seconds)
Max 50-ms moving avg. acc. (g's)	x-direction	-3.3 (0.0183 - 0.0683 seconds)	-3.2 (0.0184 - 0.0684 seconds)	-3.2 (0.0187 - 0.0687 seconds)	-3.3 (0.0185 - 0.0685 seconds)
	y-direction	-3.4 (0.0864 - 0.1364 seconds)	-3.4 (0.0901 - 0.1401 seconds)	-3.4 (0.0839 - 0.1339 seconds)	-3.6 (0.0930 - 0.1430 seconds)
	z-direction	-3.3 (0.5140 - 0.5640 seconds)	-3 (0.5468 - 0.5968 seconds)	2.6 (0.5462 - 0.5962 seconds)	-2.4 (0.5397 - 0.5897 seconds)
Maximum Angular Disp. (deg)	Roll	8.4 (0.7132 seconds)	8.2 (0.7268 seconds)	8.6 (0.7947 seconds)	8.2 (0.7326 seconds)
	Pitch	-3.5 (0.7330 seconds)	-3.7 (0.6669 seconds)	-3.6 (0.7947 seconds)	-3.8 (0.5592 seconds)
	Yaw	-35.5 (0.5842 seconds)	-34 (0.6098 seconds)	-33.6 (0.6158 seconds)	-35 (0.5559 seconds)

The project team also performed supplemental evaluations to determine the performance threshold for a 12-inch-tall timber railing regarding successful containment and redirection of a 5,000-lb pickup. Although these lower profile railings do not meet MASH TL-1, it was of interest to the TAC to determine the limits for impact conditions at which these low-profile barriers will successfully contain and redirect the 2200P vehicle (i.e., 5,000-lb quad-cab pickup). This information may be useful for bridge owners in assessing "risk" for bridge design options (e.g., to widen bridge or not) for these low-profile railing/curbs which are used on many narrow, single-lane bridges to support farming and logging industry. These bridges often include low-profile curbs with no other railing features. Taller curbs or railing designs are not applicable in these cases because they would make direct contact with the oversized loads which are often hauled across these bridges.

For the evaluations, the WVBR bridge rail was modified by reducing the height of the scupper blocks to achieve an overall height of 12 inches for the bridge rail measured from the deck surface to the top of the railing, as illustrated in figure 14. The 4-by-12 plank deck (i.e., the critical deck type) was also used for these evaluations.



Source: FHWA

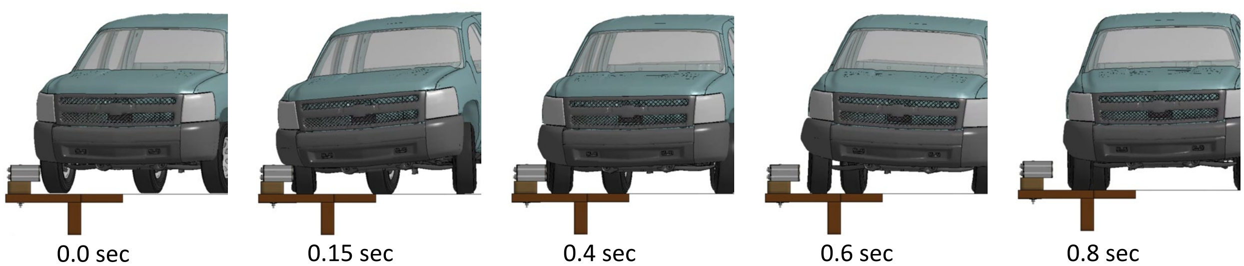
Figure 14. Graphic. Engineering drawing for 12-inch timber curb/rail used in analysis.

Due to the low impact severity for these cases, the only safety metric of concern was the tendency for the vehicle to override the barrier. Table 11 shows the results of each analysis case indicating a pass (i.e., vehicle containment) or an override condition. Sequential views for select analysis cases are shown in figures 15 through 20. Additional results including sequential views are provided in the Guide.

Table 11. Summary of results for low-profile curb/rail cases.

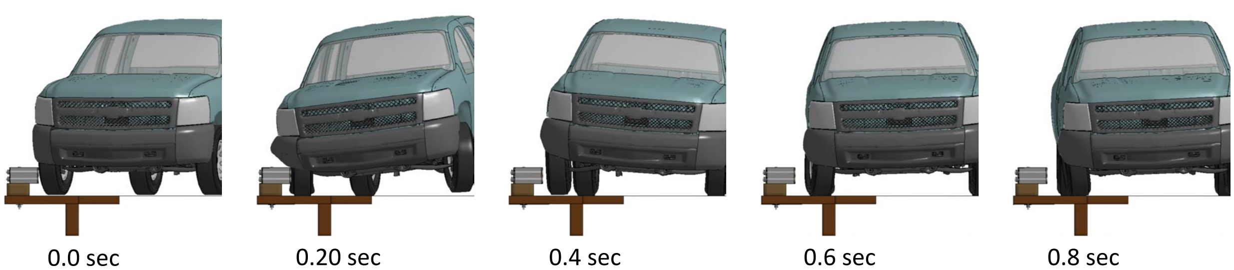
Impact Speed	Impact Angle	
	15 Degrees	**25 Degrees**
10 mph	Pass Expected*	Partial Redirection/ Partial Override
15 mph	Pass	Complete Override
20 mph	Pass	Complete Override
25 mph	Borderline Pass	Complete Override Expected*
30 mph	Override Expected*	Complete Override Expected*

* *Analysis not conducted*



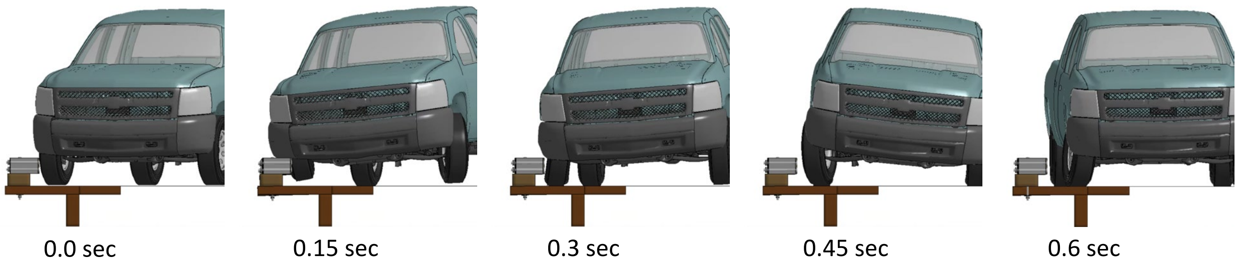
Source: FHWA

Figure 15. Graphic. Sequential views of impact at 15 degrees and 15 mph.



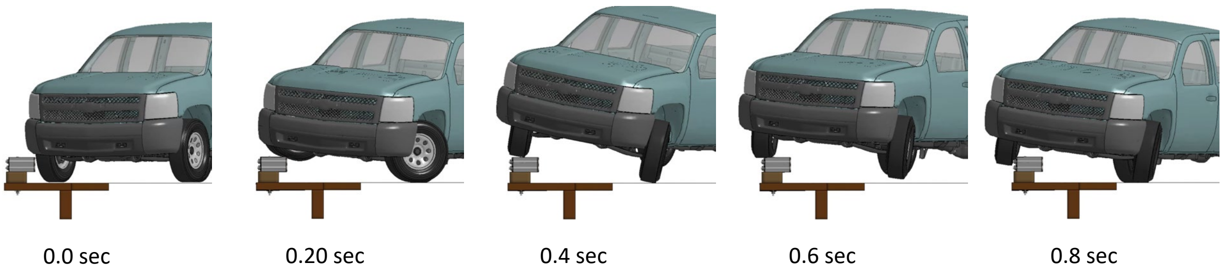
Source: FHWA

Figure 16. Graphic. Sequential views of impact at 15 degrees and 20 mph.



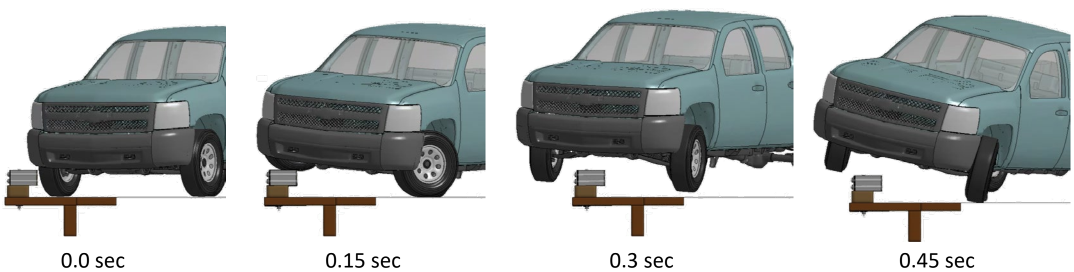
Source: FHWA

Figure 17. Graphic. Sequential views of impact at 15 degrees and 25 mph.



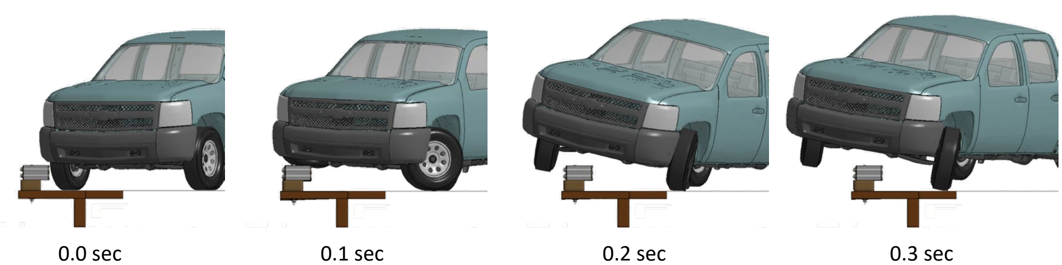
Source: FHWA

Figure 18. Graphic. Sequential views of impact at 25 degrees and 10 mph.



Source: FHWA

Figure 19. Graphic. Sequential views of impact at 25 degrees and 15 mph.



Source: FHWA

Figure 20. Graphic. Sequential views of impact at 25 degrees and 20 mph.

The results indicated that the 12-inch-tall curb/railing would successfully contain and redirect the 5,000-lb pickup at impact speeds of 20 mph or less when the impact angle was less than or equal to 15 degrees. For the 15-degree impact cases that resulted in successful containment of the vehicle, the maximum impact severity was 4,475 ft-lb. The analyses also indicated that the 12-inch-tall curb/railing would successfully contain and redirect the pickup when impact speeds were less than 10 mph for an impact angle of 25 degrees. For the 25-degree impact cases that resulted in successful containment, the maximum impact severity was 2,983 ft-lb. It was therefore concluded that impact severity does not appear to correlate well with the pass/fail conditions for the low-profile barrier.

This is likely because the stiffness and strength of the barrier were not governing factors for successful containment of the vehicle in the evaluations. Success or failure was predominately determined by the tendency for the tire to climb the barrier during impact, which occurred more prevalently at higher impact angles due to the tire tread having more contact with the railing. It is expected that other tire related factors will also affect the tendency for vehicles to override low profile curb/railing barrier. For example, larger diameter tires and/or tires with lower inflation pressure would likely have higher tendency to override. The project team suggests that additional testing and analysis be performed to further validate the steer response for the model and to also verify the results presented herein regarding the performance limits of the 12-inch-tall curb/railing.

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