Final Report:
Recommended Guidelines for the Selection of Test Levels 2 through 5 Bridge Railings
NCHRP 22-12(03)

February 10, 2014

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INTRODUCTION

There has been interest for several decades in developing selection guidelines for the multiple performance, service or test levels for bridge railings in the roadside safety community. Since bridges cross over large spans of space they often cross significant features such as busy transportation corridors. In addition, bridges carry heavy vehicles sometimes with dangerous cargos, such as fuel and hazardous chemicals. The consequences to public safety of a heavy truck penetrating through or rolling over a bridge railing or a passenger vehicle vaulting a bridge railing present additional risks not considered for crashes with other types of roadside barriers.

Numerous bridge railings have been designed and crash tested in the past several decades according to one of the several multiple test level approaches so there are a wide variety of different test level bridge railings available. What has never been established, however, are the criteria for selecting when a higher test level railing is needed based on the specific traffic and site characteristics of individual bridges. The American Association of State Highway and Transportation Officials (AASHTO) Roadside Design Guide (RDG) and LRFD Bridge Design Specification recognize the multiple test level approach but give only very general guidance about why a higher test level bridge railing might be used. [AASHTO06] At present, highway agencies must make decisions on which test level is appropriate for each site on an *ad hoc* basis.

The objective of this project was to develop proposed selection guidelines to assist bridge engineers and highway designers in selecting an appropriate test level for bridge railings based on specific site and traffic conditions. The focus of the study was on TL2 through TL5 railing. TL1 bridge railings involve very low volume and low speed applications which are not widely encountered and are generally not considered practical except in some special situations like park roads. At the other end of the spectrum, TL6 bridge railings are presumably intended for locations where the severity of a penetration or rollover would be exceptionally catastrophic. A TL6 bridge railing would be warranted to maintain public support for the highway project, even if the barrier is not necessarily cost effective. In addition, there is only one crash tested TL6 bridge railing available at this time. It requires specially designed deck details to support the impact loads and additional dead load of the barrier. The vast majority of bridge railings that will be practical for use are, therefore in the TL2 through TL5 range.

The basic approach used in this project was a risk-based approach where the frequency and severity of crashes with bridge railings are estimated and the risk of observing a serious or fatal injury crash calculated. The third version of the Roadside Safety Analysis Program (RSAP) was developed to perform cost-benefit analysis, but was expanded during this research effort to preform risk-analyses as well. This report documents the research conducted to populate the RSAP database, run the RSAP simulations, and the resulting selection guidelines for the selection of MASH TL2 through TL5 bridge railings. The literature reviewed and a survey of practitioners are presented in the first several chapters, the data gathered and the analysis conducted is presented alongside a discussion of the decisions made throughout the research in subsequent chapters. Finally, the proposed selection process and selection tables to accompany the process are presented at the end of the document with accompanying discussion and alternative selection tables for use in the establishment of policy. The recommendations are presented in their entirety in Appendix B for a quick reference.

LITERATURE REVIEW

According to a Federal Highway Administration (FHWA) sponsored study in the 1980's, there are about 500,000 bridges in the U.S., about half of them on the National Highway System. [Mak83] Mak also found that the fatal crash rate was about three times higher on bridges than on similar road segments. [Mak83] One of the consequences of Mak's findings was a steady evolution in the guidelines for the design and testing of bridge railings. In the 1980's, bridge railings did not have to be crash tested and many bridge railings were found to be structurally inadequate. Persistent research and testing in the past several decades has provided many improved bridge railings with crash-test demonstrated impact performance.

The increase in miles of public road and miles of bridges have increased at about the same pace over the last ten years with bridges consistently remaining approximately 0.40 percent of the total mileage as shown in Figure 1.[FHWA12a] The mileage of urban bridges, however, is increasing at a faster pace than rural bridges (Figure 2) indicating that more bridge rail penetrations in more sensitive urban areas may become more common in the future. [FHWA12a]

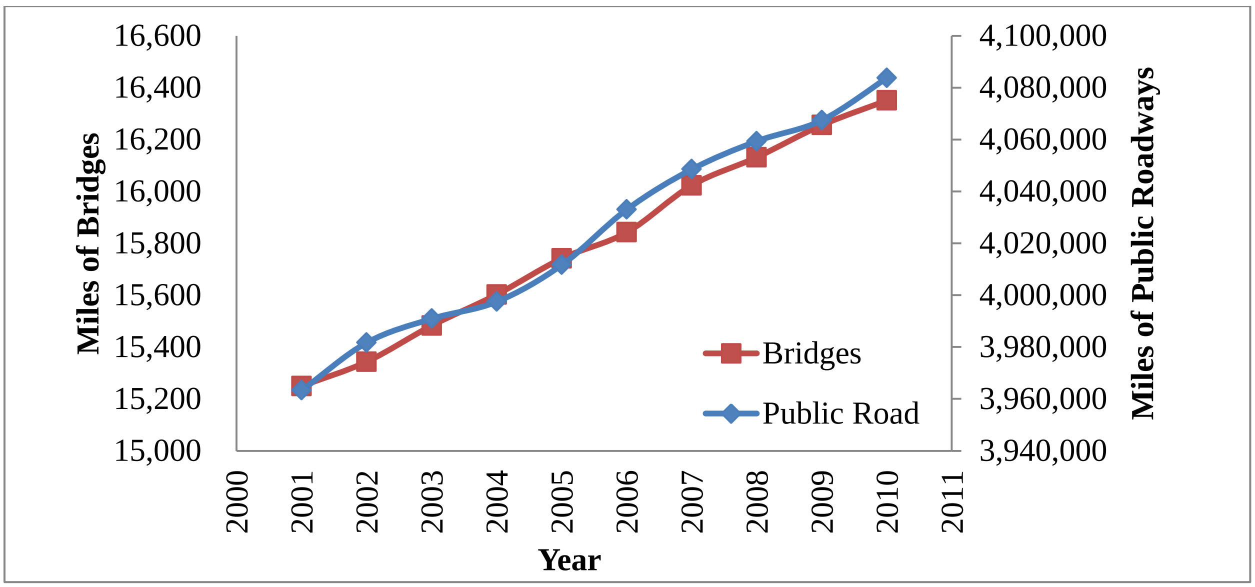


Figure 1. Mileage of Public Roads and Bridges.

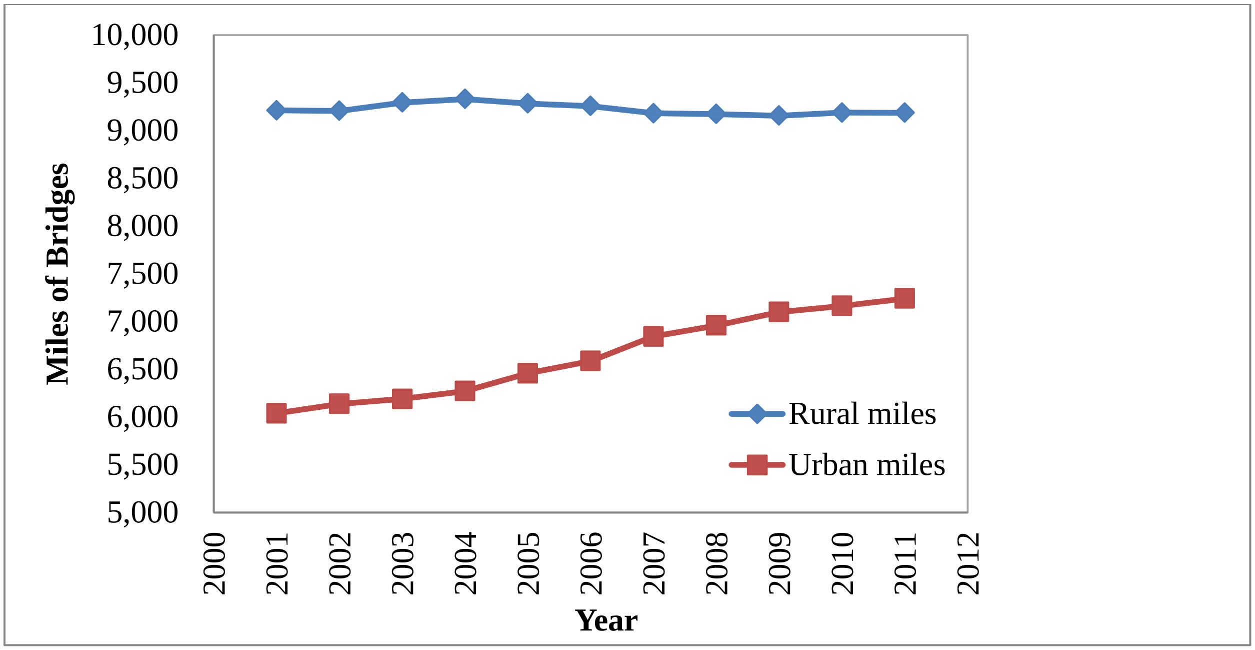


Figure 2. Miles of Urban and Rural Bridges.

Exemplar Crashes

The rare occasions when a bridge railing fails to restrain an errant vehicle often results in dramatic crashes. Such crashes have the potential to involve loss of life, the involvement of multiple vehicles, extensive property damage and significant traffic delays. While they do not occur often, when they do occur they nearly always are reported in the news media and demand public attention. The purpose of this next section is to review some bridge railing crashes that have appeared in the media and that have been investigated by the National Transportation Safety Board (NTSB) in order to gain a perspective on both the causes and consequences of bridge railing failures. An appreciation of the causes and consequences will be vital to properly selecting appropriate test levels based on the traffic, and the operational and site conditions of a particular bridge.

Crashes in the Media

St. Petersburg, Florida, 2001

On January 1st of 2001 a single-unit truck was travelling south on I-275 across a bridge over 54th Avenue South in St. Petersburg, Florida when the truck struck the bridge railing. The impact fractured the concrete railing and the front axle was separated from the truck allowing the axle and pieces of the concrete railing to fall into the bed of a pickup travelling on the roadway below. The axle-less vehicle continued downstream where it straddled the concrete barrier and vaulted over the top, coming to rest in the lanes of 54th Avenue South below where it struck a bus. The driver of the bus was killed and two other people were injured in the crash. Concrete and vehicle debris struck at least three vehicles in addition to the bus. The barrier damage from the initial impact with the single-unit truck is shown in Figure 3. The skid marks in Figure 3 suggest a relatively high impact angle. Other information from the scene suggest the single-unit truck left the roadway on the right and then crossed two lanes before striking the bridge railing on the left. The final positions of the single-unit truck and bus are shown in Figure 4. [Alberson04]

Figure 3. Damage to bridge railing, St. Petersburg, FL...

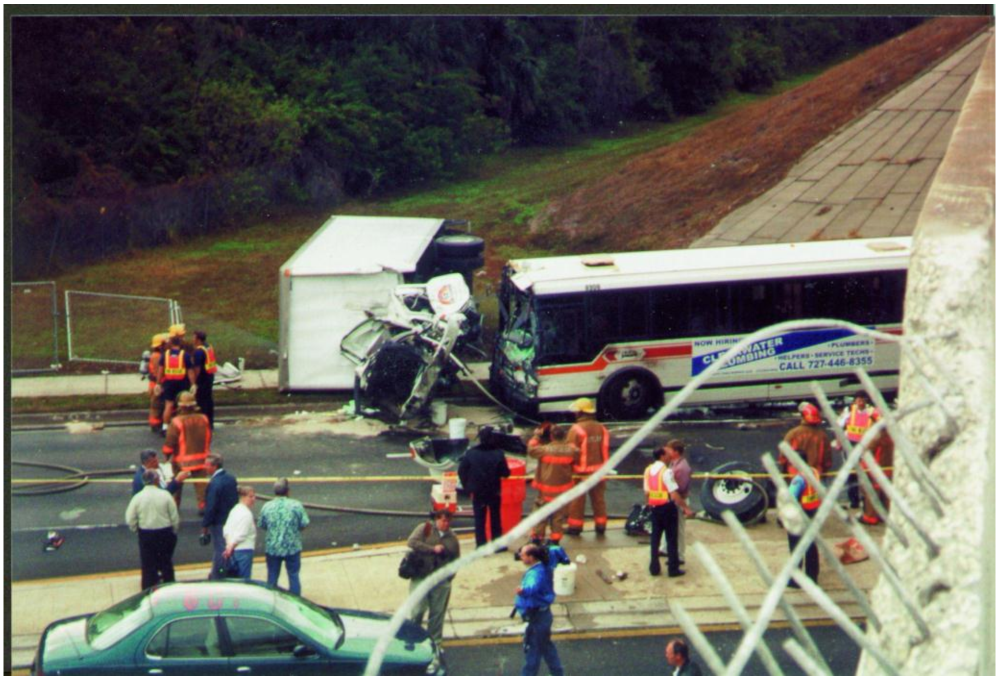


Figure 4. Final position of the single-unit truck and bus, St. Petersburg, FL.

Glenmont, New York, 2007

Figure 5 shows an example of a tractor trailer truck that penetrated a bridge railing in New York State that was clearly not designed to restrain heavy vehicles. There were apparently no serious injuries in the crash but Figure 5 illustrates the potential dangers for facilities beneath the roadway when a heavy vehicle penetrates the bridge railing.

Figure 5. Tractor trailer truck penetration of bridge rail, Glenmont, NY.

Wiehlthal Bridge, Germany, 2004

There have even been a few instances where a truck crash caused major structural damage to the bridge requiring the replacement of the bridge itself. Figure 6, for example, shows the result of a truck crash in 2004 on the Wiehlthal Bridge in Germany. On August 26, 2004 a passenger car collided with a fuel tanker truck on the Wiehlthal Bridge on the A4 motorway between Cologne and Olpe, Germany. The fuel tanker truck, which was carrying 8,500 gallons of fuel, penetrated the bridge railing, fell 100 ft. and then burst into flames, killing the driver. The flames burning under the bridge structure caused the steel to deform and lose its load-bearing capacity resulting in the closure of the bridge and the need to completely replace it. Temporary repairs to restore traffic cost the equivalent of $42 million. The total crash cost has been estimated at nearly $400 million; certainly one of the most expensive traffic crashes in history.

Figure 6. Tractor trailer truck which penetrated the Wiehlthal Bridge, Germany. [Wiehltal04]

San Francisco, California, 2009

Figure 7 shows the result of a tractor trailer truck penetrating the bridge railing on the Bay Bridge between Oakland and San Francisco, California in November 2009.[Zimbio09] The truck fell 200 ft. onto an island, killing the driver. According to news reports, traffic was stopped on this bridge, which carries 250,000 vehicles/day, for over nine hours. Such long delays affecting such large numbers of people create a significant travel delay cost. These costs are not captured in the usual crash cost data.

Figure 7. Result of a truck penetrating the Bay Bridge, San Francisco, CA.

Boston, Massachusetts, 2007

On Tuesday April 2nd, 2007 a tractor trailer truck was traveling on the entrance ramp to I-93 south in Charleston Massachusetts. [WBZ07; Globe07; Boston07] The truck struck a 30-inch tall concrete safety shape barrier on a horizontally curved elevated ramp and rolled over the barrier resulting in a 70 foot free fall onto another elevated on-ramp below. The tractor trailer struck a luminaire and fell on to a sport utility vehicle traveling on the ramp below.

While the driver of the sport utility vehicle and the truck driver were both hospitalized with non-life threatening injuries, the situation could have been much worse. The crash happened at off-peak travel hours so there were relatively few vehicles travelling on the ramp below. If the crash had occurred during the peak travel hours, the truck would have fallen onto many more vehicles. Also, the ramp the tractor trailer truck fell onto was also an

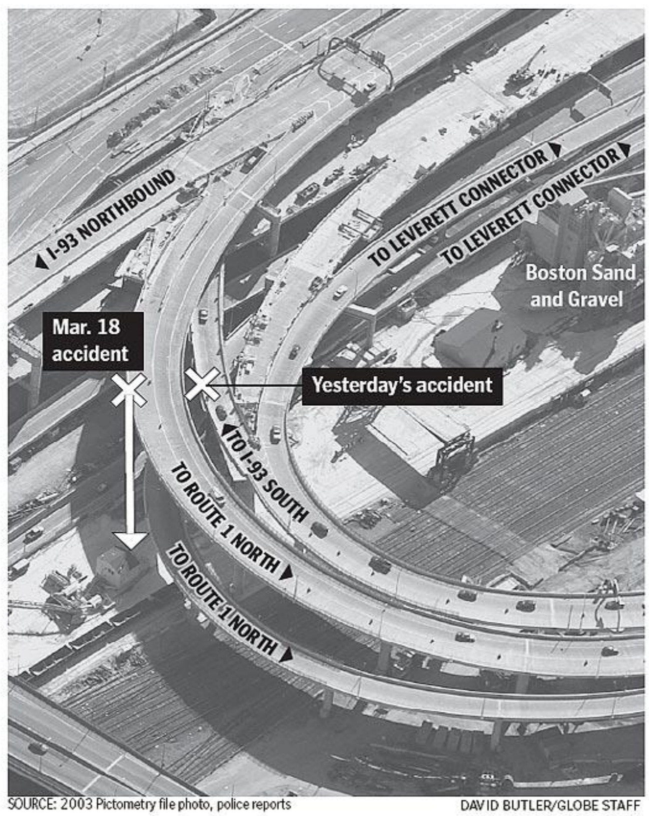


Figure 8. Location of two heavy vehicle crashes where a bridge rail was penetrated, Boston, MA. [WBZ07]

elevated ramp that crossed over the passenger rail lines to North Station. With only slightly different impact conditions the truck could have easily struck vehicles on the lower ramp and continued to fall onto the rail line where it may have been struck by a passenger train. The consequences of such a crash could have jeopardized the health and safety of hundreds of people in this highly congested heavily urbanized area especially if the truck had been carrying fuel or a hazardous cargo. This particular crash, unfortunately, was not an isolated event since a very similar crash occurred at essentially the same location only a few months before.

The bridge railing at this site was essentially a TL3 F-shape concrete barrier. According to the project design documentation, the railing was originally intended to have a metal top rail which would have made it a TL4 bridge railing. The TL4 railing appears to have been "value engineered" out of the project to save funds. The crash history at this site suggests that a TL3 railing was perhaps not the best choice for this heavily travelled roadway with such potential for catastrophic crashes.

Amesbury, Massachusetts, 2011

A fatal bridge railing crash occurred on I-95 in Amesbury, Massachusetts involving a small passenger car in 2011. [Eagle-Tribunne11] While driving northbound on I-95 on the approach to the Whittier Bridge over the Merrimack River in Amesbury, Massachusetts, a 64-year old man lost control of his 2002 Toyota Camry and struck the bridge railing. The vehicle vaulted over the bridge railing and fell 100 feet into the Merrimack River where it sank another 20 feet to the river bottom. An extensive search and rescue effort was required to locate the vehicle and driver but, unfortunately, the driver died as a result of the crash. The bridge railing at this site appears to be a 32-inch tall concrete safety shape. There appear to have been tall snow banks in front of the barrier that may have contributed to the vehicle vaulting over the railing. This crash points out that while heavy vehicles have more energy and are often associated with bridge rail penetrations and rollovers, passenger vehicles can also vault or roll over bridge railings.

Avon, Colorado, 2012

Sunday, May 13, 2012 at 9:45 AM in Avon, Colorado, a tandem trailer truck lost control in the I-70 westbound lane and penetrated the Avon Road overpass. One of the trailers landed on a Honda CRV traveling south on Avon Road, crushing the driver's side of the vehicle. Both of the occupants of the Honda CRV were unharmed. However, the truck driver was fatally injured.

A witness said "…the rear trailer had become detached and was traveling at a high rate of speed, both airborne and facing backwards when it flew off the highway and landed on the CRV. Moments later, the cab and trailer to which it was attached also tumbled off the overpass" leading to an explosion and fire (Figure 9). Figure 10 shows the w-beam rail in place at the time of the crash. [Avon12]

Figure 9. Truck in flames under bridge after penetrating the rail, Avon, CO. [Avon12]

Figure 10. Damaged W-beam bridge rail and final position of truck on local road under bridge, Avon, CO. [Avon12]

Syracuse, New York, 2012

On July 22, 2012, the front cab of a tractor-trailer penetrated a concrete bridge rail in Syracuse, New York and dropped 25 feet. The two occupants of the cab were injured, but responsive after the crash. The tractor-trailer was traveling north on Interstate 81 and penetrated the rail between Erie Boulevard East and Water Street. The frame, motor and the axle remained hanging from the bridge (Figure 11, Figure 12). About 100 gallons of diesel fuel was spilled.[Syracuse12]

Figure 11. Cab of tractor-trailer penetrated rail and fell to road below, trailer remained on bridge, Syracuse, NY. [Syracuse12]

Figure 12. Site of bridge railing penetration, Syracuse, NY. [Syracuse12]

Montreal, Quebec, 2011

On Wednesday, December 28th, 2011 a pickup truck traveling west on the Sainte Anne de Bellevue Road near the entrance to Highway 20 lost control and left the roadway. The vehicle flipped off an overpass and fell onto railway tracks, where it was then hit by a train (Figure 13). Both occupants of the pickup truck died in the crash. Police speculated poor visibility and slippery roads may have played a role. [CBC01] The section of roadway that runs over the railroad tracks had a w-beam guardrail as the only barrier, as can be seen in Figure 14, however, after the crash occurred, Transport Quebec installed a concrete barrier at the location.

Figure 13. Truck Came to Rest on Railroad Tracks, Montreal, Quebec. [GAZ01]

Figure 14. W-beam Guardrail Roadside Barrier, Montreal, Quebec. [Google Earth]

Avellino, Italy, 2013

On July 29th, 2013, a bus carrying around 50 people hit several vehicles before penetrating a bridge rail and falling 98 ft. into a gorge near Avellino, Italy. (Figure 15) At least 38 people were killed in the crash, with an additional 10 people injured. [BBC01] The rail type is unknown.

Figure 15. Removal of Bus from the Bottom of Gorge, Avellino, Italy. [BBC01]

Beaverton, Oregon, 2012

On Saturday, November 24, 2012 in Beaverton, Oregon a crash resulted in a pickup truck hanging from the SW Denney bridge railing over HWY 217. The driver was the only occupant (Figure 16). Fire and rescue personnel responded and secured the truck to a fire engine to keep the truck from falling off the bridge. A fire engine equipped with a basket was used to get the driver out of the truck safely. [KATU01] The bridge railing appears to be a PL2 system named the "Foothills Parkway Bridge Railing," approved under AASHTO Guide Specification for Bridge Railings. The railing as it appeared before the impact is shown in Figure 17.

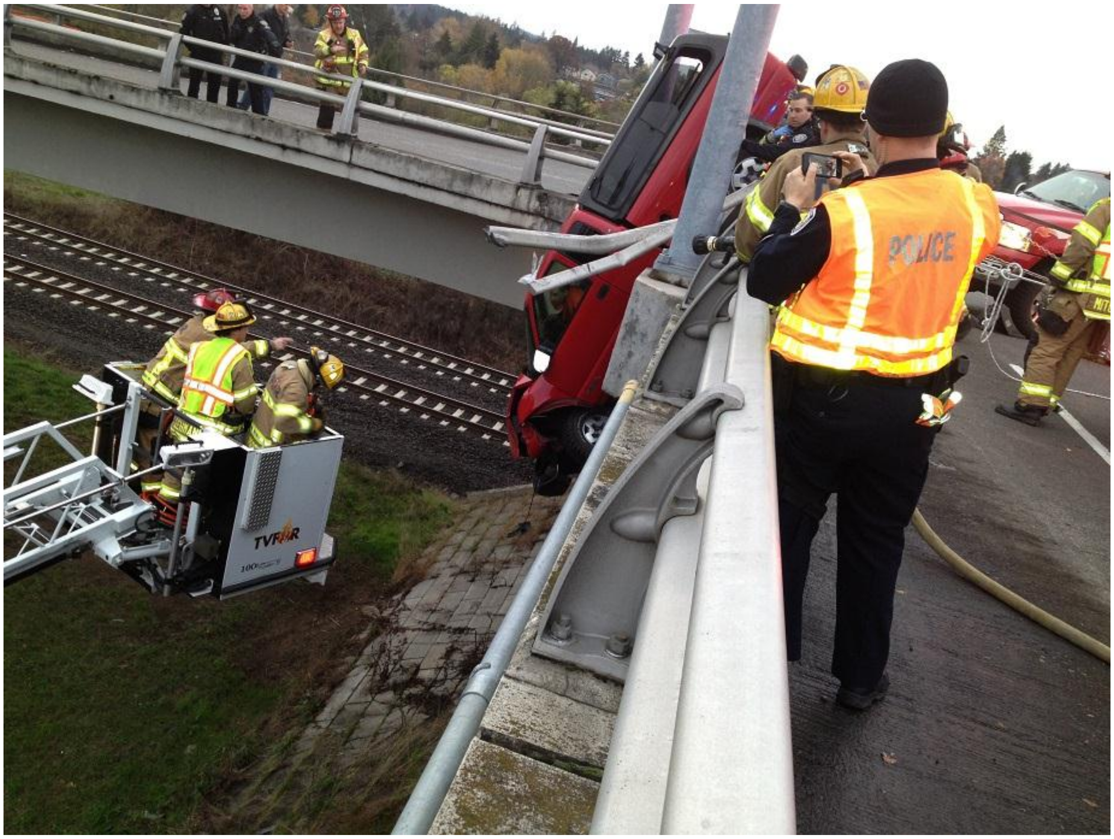


Figure 16. Pickup Truck After Rail Penetration, Beaverton, OR. [ORHER01]

Figure 17. Beaverton Bridge Rail Before Crash, Beaverton, OR. [Google Earth]

Bronx, New York, 2012

On April 29th, 2012, a white 2004 a white Honda Pilot minivan "…was traveling southbound at about 70 mph in a 50-mph zone of the Bronx River Parkway when the 12:30 p.m accident occurred." [NYDN01] The minivan lost control while driving southbound on the Bronx River Parkway and hit the single-slope median barrier. After striking the barrier, the minivan veered to the right, crossed three lanes of traffic, struck the curb on the right side of the road and vaulted over an older style metal post and tube bridge rail. "The car never even touched the 4-foot-high iron guardrail, though it left traces of motor oil on the rail as it sailed over it." There were no skid marks found on the road before the first impact, however, there were skid marks leading up to where the van left the bridge. [NYDN01] The vehicle fell 60ft off of the bridge, landing on its roof in a wooded area. All seven passengers, including three children, were killed instantly. [CNN01]

Less than a year earlier, in the same section of the Bronx River Parkway, a second crash occurred. On June 4, 2011, two people were uninjured after the driver lost control of the vehicle, "… flew off a parkway overpass, and landed 22 feet below in the parking lot of a police station. The media report the 2005 Acura was speeding while traveling north in the left lane before it slammed into a concrete median barrier. "Then, it flew across both lanes of traffic, crashing through a guard rail and a chain link fence before partially landing on an officer's GMC pickup truck in the parking lot of the NYPD 12th district station in Morris Park." [WPIX01]

A picture of the approximate location of both crashes can be seen in Figure 18, courtesy of Google Earth.

Figure 18. Bronx River Parkway, near crash site, Bronx, NY. [GoogleEarth].

Grand Prairie, Texas, 2013

On August 3rd, 2013 a tractor trailer was traveling westbound on Interstate 30 in Grand Prairie, Texas when the truck veered off the right shoulder. The truck penetrated the guardrail, then struck the bridge abutment and continued down the embankment in between the guardrail and abutment until it crashed down onto State Highway 161 below. [KVUE01] The truck landed on State Highway 161 and hit a single slope concrete barrier head on, penetrating the barrier when it burst into flames, narrowly missing the I-30 westbound bridge pier (Figure 19 and Figure 20). The driver died in the crash. There were no other injuries.

While this crash appears not to be technically a bridge railing crash since the vehicle penetrated the approach guardrail prior to the bridge, it does point out the importance of guardrail-bridge railing transitions. In particular, typical w-beam guardrails are TL3 devices whereas bridge railings may be TL3, TL4 or TL5. This crash raises the question of how far in advance of the bridge railing a higher test level barrier may be needed to prevent this type of catastrophic heavy vehicle crash.

Figure 19. State Highway 161 where Truck Landed, Grand Prairie, TX. [WFAA01]


Figure 20. State Highway 161 where Truck Landed, Grand Prairie, TX. [WFAA01]

Boston, Massachusetts, 2013

On August 6, 2013, a beer delivery truck partially penetrated the bridge rail on I-93 North resulting in the cab dangling three feet beyond the bridge rail as shown in Figure 21. The driver and the passenger were uninjured. Traffic on I-93 was significantly slowed, with one commuter who was stuck in traffic noting: "I've gone about 100 feet in an hour." [WBZ01] This crash also disrupted commuter traffic on the Orange MBTA subway line since the commuter rail parallels the highway.

Less than a week later and only a few feet away, on August 9, 2013 a Cadillac struck a single unit truck on Interstate 93 northbound in Boston, Massachusetts sending the single unit truck into the bridge rail. The truck vaulted over the rail and landed on the Exit 26 southbound ramp. The rail type can be seen in Figure 22. Despite falling approximately 40 ft. before crashing onto the roadway below, the driver of the truck sustained only minor injuries. [Patriot01, WCVB01]

Figure 21. Boston Beer Truck Penetration, Boston, MA. [WBZ01]

Figure 22. Close-Up of Bridge Rail Type (upper left) and Exit 26 Ramp Truck, Boston, MA. [Google Earth]

Buellton, California, 2012

On January 12, 2012 on Highway 101 in Buellton, California, a trailer truck drifted out of its lane and sideswiped a car in the lane next to it. [NBC01] After repeatedly pushing the passenger car into the bridge rail, the wheels of the trailer ran over the car and broke through the rail. The truck and trailer fell down into the creek below and burst into flames, killing the driver. (Figure 23) The passenger car was left hanging off the bridge, still containing the driver and two children. Fortunately, a unit of Navy Seabees was on the adjacent bridge transporting a heavy-duty forklift which they used to stabilize the vehicle while emergency personnel extracted the passengers. (Figure 24) The driver and one child suffered serious injuries while the second child was unharmed.[MOUK01]

Figure 23. Truck in Final Resting Position, Buellton, CA . [MOUK01]

Figure 24. Rescue Workers Stabilizing Car and Freeing Passengers Buellton, CA. [MOUK01]

Galesburg, Illinois, 2013

On September 7, 2013, an SUV was involved in a single-vehicle run-off road crash. The SUV failed to negotiate a curve on U.S. Route 150 on the Gates Bridge. The vehicle went over the guardrail, landed on the railroad tracks below the bridge, and came to rest in a wooded area on the south embankment. One person who was partially ejected from the vehicle was killed. Another passenger was injured in the crash, but the extent of the injuries was not released. [PJSTAR01] A Google Earth image of the pre-crash area is located in Figure 25.

Figure 25. Gates Bridge, Galesburg, IL. [Google Earth]

Williamsburg, Kansas, 2012

A 17-year-old boy with a provisional license was driving a Freightliner motor home that was pulling a trailer on Interstate 35 when it went off the road, struck a guardrail and crashed through a bridge rail and fell into a ravine on April 1, 2012. (Figure 26) It is not clear from news reports if the guardrail was penetrated first or if the bridge railing was penetrated after the guardrail redirected the vehicle. Of the 18 passengers, five were killed and the remaining 13 were all injured, two critically. [MOUK02]

Figure 26. Recreational Vehicle Location in Ravine, Williamsburg, KS. [MOUK02]

On March 23, 2013, it was announce that the National Transportation Safety Board (NTSB) would be taking over the investigation of this crash, including looking into laws that allowed the 17-year-old boy with a provisional license to drive the 57,000 lbs vehicle.

Crashes Investigated by the National Transportation Safety Board

The National Transportation Safety Board (NTSB) investigates and determines the probable cause of "significant crashes" on highways and other modes of transportation with the goal of promoting transportation safety and preventing future similar crashes. In total, NTSB investigates approximately six highway crashes per year, each investigation lasting approximately 20 months.

NTSB crash investigation teams vary in size from three or four to more than twelve specialists who routinely handle investigations within their specialized field (i.e., rail, highway, marine and pipeline). Highway crash teams include specialists with backgrounds such as a truck or bus mechanical expert, a highway engineer, a weather specialist, a human performance specialist, and survival factors specialist. The team is led by an Investigator-in-Charge.

The following sections contain brief summaries of crashes involving bridge railings which have been investigated by the NTSB since the mid-1970's. While from a statistical point of view, these crashes are anecdotal, they do serve to point out important features of catastrophic crashes involving bridge railings.

Fort Sumner, New Mexico, 1972

On December 26, 1972, a school bus transporting 34 people was traveling westbound while a tractor-trailer truck transporting cattle was eastbound on US-60 near Fort Sumner, New Mexico. [NTSB74] As the truck approached a narrow bridge, the driver swerved to the right after seeing approaching headlights that appeared to be on his side of the road. The truck struck a crash cushion at the entrance to the bridge and the right-rear wheel of the trailer mounted the curb on the bridge. The tractor "snagged" the bridge railing and rotated, mounting the curb and causing the trailer to jackknife. The bus collided with the jackknifed trailer in the westbound lanes. Nineteen people in the bus were killed and 15 others sustained a variety of injuries. As a result of this crash, the NTSB recommended that the FHWA "*expedite a program to improve, where feasible, substandard bridge-rail systems on existing bridges to increase resistance to pocketing or penetration by impacting vehicles of all classes and redirect those vehicles. Research, including crash testing, should also be expedited to develop criteria for mandatory standards for bridge-rail and guardrail designs for new bridge construction* (H-74-7)." [NTSB74]

Nashville, Tennessee, 1973

On July 27, 1973 while traveling through a morning fog a car carrying nine people penetrated the bridge railing on the Silliman Evans Bridge in Nashville, Tennessee and fell 65 feet to the ground below. Seven passengers and the driver were killed. The barrier on the bridge was apparently a type of box-beam barrier mounted on a nine-inch curb. As a result of this crash the NTSB recommended that the FHWA "*establish national performance standards, including dynamic testing procedures, for bridge rail systems. Such standards should extend performance criteria to include impacts by heavy vehicles and should improve performance characteristics for impacts by all classes of vehicles. The establishment of these standards should be of high priority and compliance should be mandatory for all new bridge rail systems used on public roadways* (H-74-18)." [NTSB74]

Siloam, North Carolina, 1975

In the morning of February 23, 1975, an automobile was traveling in a heavy fog when it penetrated a timber bridge railing and struck a structural member on the Yadkin River Bridge near Siloam, North Carolina. [NTSB76] The bridge was a through-truss bridge so the penetration of the bridge railing allowed the vehicle to damage a vital structural component of the bridge. As a result of the collision, the bridge collapsed and fell into the river. Six additional vehicles drove into the river in the next 17 minutes resulting in four people being killed and 16 others being injured. As a result of this crash the NTSB recommended that the FHWA develop and publish "*guidelines for the structural retrofit of bridge railings on existing bridge structures to protect vital structural members from impact by vehicles*." [NTSB76]

Martinez, California, 1976

On May 21st, 1976 a school bus was travelling on I-680 near Martinez, California with 52 people on-board when it struck a bridge railing on an off-ramp. The bus rolled over the bridge railing of the curved bridge, landing on its roof. Twenty nine people were fatally injured in the crash. [NTSB77a] The bridge railing and the integrated curb were cited by the NTSB as one of the contributing factors to the crash.

As a result of this crash, the NTSB made three recommendations that dealt with various aspects of bridge railing design and placement. The three recommendations were:

H-77-12: "*Develop bridge railing designs that will meet performance standards to be established by the FHWA for various classes of vehicles and that will be sufficient in number to meet the various state requirements with regard to climatic and other physical conditions that affect the operation and maintenance of a roadway system. Such bridge barrier railing designs should be available to states that do not desire to develop their own designs in accordance with mandatory performance standards issued by the FHWA.*" [NTSB77b]

H-77-13: "*Investigate through dynamic crash testing and analytical procedures the effects of various geometric configurations and adjacent roadway surfaces on the performance of traffic barrier rail systems. The investigation should also consider how maintenance practices or the lack of maintenance affects the performance of the barrier rail systems*." [NTSB77c]

H-77-14: "*In cooperation with the states, establish priority guidelines for improving, through modification or retrofit, the performance of existing traffic barrier rail systems at bridges. Consideration should be given in the priority guidelines to the potential for multi-fatality accidents involving high occupancy vehicles such as buses*. [NTSB77d]

Houston, Texas 1976

A tractor-trailer truck hauling over 7,500 gallons of anhydrous ammonia was traveling on an elevated ramp between I-610 and US-59 in Houston, Texas on May 11, 1976. [NTSB77e] At this location, US-59 is at the ground level and I-610 is elevated over it. The ramp between I-610 and US-59 passes between I-610 above and US-59 on the ground below. While negotiating the ramp, the tractor trailer truck began to roll due to the horizontal curvature of the ramp and the truck's speed. The truck penetrated the bridge railing and fell about 15 feet onto US-59. As the truck fell it also struck and sheared off a column supporting the elevated portion of I-610 above. The truck and trailer became detached during the crash and the trailer broke into several parts allowing the rapid escape of the ammonia into the atmosphere. Twelve automobiles were damaged by flying debris from the tractor and trailer as it crashed onto US-59. The driver of the truck was fatally injured in the crash and five people were killed and 178 people injured due to breathing the ammonia gas that escaped from the ruptured trailer.

I-610 was designated in 1970 as a hazardous materials route by the City of Houston and all vehicles transporting hazardous materials through the city were restricted to this route. I-610 is an elevated five-lane highway near the crash site. The ramp onto US-59 where the crash occurred consists of two lanes arranged in an interconnected three, six and 12 degree compound curve; the crash occurred on the third curved section (i.e., the 12 degree curve). The bridge railing was an oval pipe section 33.5-inches high, mounted on five-inch wide, ¾-inch thick steel bars. The support bars were bolted behind a 14-inch wide, 12-inch tall curb. The crash destroyed 94 feet of bridge railing, caused damage to the bridge deck, a column supporting the I-610 overpass was sheared off and guardrails on US-59 were damaged.

Figure 27. Hazardous material truck crash near Houston, Texas in 1976. [NTSB77e]

As a result of this crash, the NTSB recommended that the FHWA "*in consultation with State and local governments, establish highway design criteria for the selection, location and placement of traffic barrier systems that will redirect and prevent penetration when struck by heavy vehicles. The criteria for preventing vehicle penetration should consider the human exposure to injury and the effects of hazardous cargo that could result from barrier penetration* (H-77-5)." [NTSB77e]

Elkridge, Maryland, 2004

On January 13, 2004 a tractor tanker-trailer truck was hauling 8,800 gallons of gasoline southbound on I-895 near Elkridge, Maryland. [NTSB09a] As the tanker truck approached the curved and elevated I-95 overpass it entered the right shoulder and struck the guardrail and continued on to strike the attached bridge railing. The truck and trailer mounted and vaulted over the bridge railing falling 30 feet onto the northbound lanes and median of I-95. The NTSB estimated the vehicle's speed did not exceed 49 mi/hr. An explosion and large fire resulted from the tanker truck striking the ground and four vehicles travelling northbound on I-95 drove into the conflagration. Four of the five vehicle operators were fatally injured in the crash.

The barrier on the I-95 overpass, shown in Figure 28, was a 32-inch tall concrete safety shape bridge railing installed adjacent to a four-ft. shoulder on the overpass. The overpass was curved to the left which promoted the vehicle rolling over the barrier while the driver was trying to regain control by steering to the left. The guardrail transition to the bridge rail may also have been a factor in the crash.

While the NTSB was not critical of the choice of the bridge railing at this location, the crash site did involve some of the risk factors cited by the RDG for higher test level bridge railings. The horizontal curve had a radius of about 954 feet. I-95, the road the bridge crossed over, had an ADT of 189,750 in 2004. I-895 had an ADT of 13,350 in 2004 with 5.5 percent single unit truck classes 4 through 7 and 3.2 percent combination trucks classes 8 through 13 for a total percent trucks of 8.7.

Figure 28. Crash site in Elkridge, MD where a fuel truck penetrated a concrete bridge railing. [NTSB09a]

The 32-inch tall concrete safety shape is probably the most common Report 350 TL4 bridge railing in use today. The curved alignment of the roadway, the hazardous material being transported and the fact that the overpass was crossing a very heavily travelled and important interstate magnified the importance of the bridge railing at this particular location.

Huntsville, Alabama, 2006

On November 20, 2006 at about 10 a.m. a school bus with 40 students onboard was traveling westbound in the left lane of an elevated ramp portion of I-565 in Huntsville, Alabama. [NTSB09b] A 1990 Toyota Celica was following the bus and apparently moved into the right-hand lane and accelerated in order to pass the bus on the right. As the Toyota was abreast of the bus it began to "fishtail" and the driver lost control, veering to the left and striking the right-front tire of the school bus. Both vehicles swerved to the left and struck a 32-inch tall concrete bridge railing on the left side of the ramp. The school bus climbed up onto the bridge railing and travelled about 117 feet before completely rolling over the railing and falling about 30 feet below onto a dirt and grass area underneath the ramp shown in Figure 29. The crash resulted in four fatalities, 17 serious injuries, 17 minor injuries and three bus occupants were uninjured. The bus driver was ejected in the initial crash and four passengers were either fully or partially ejected when the bus struck the ground below. The Toyota did not penetrate the bridge railing and came to rest against the bridge railing. The driver and passengers of the Toyota were not injured in the crash.

Figure 29. Final rest position of a school bus that penetrated a concrete bridge railing near Huntsville, AL in 2006. [NTSB09b]

With respect to highway design issues, the NTSB noted that the bridge railing as a 32-inch high Report 350 TL4 concrete safety shape installed adjacent to a four-foot left shoulder. The bus was travelling no more than 55 mi/hr, it struck the railing at 9-10 degrees and its gross empty weight was 17,700 lbs so the impact conditions were not extraordinary in comparison to the standard Report 350 TL4 test (i.e., 18,000-lbs single unit truck striking the barrier at 15 degrees and 50 mi/hr). The NTSB concluded that the Toyota restricted the bus from moving back into its lane and essentially held the front of the bus to the railing until it eventually rolled over it.

Sherman, Texas, 2008

On August 8th, 2008 at about 12:45 a.m. a motorcoach with 55 passengers and a driver were travelling at about 68 mi/hr northbound in the right-hand lane of the four-lane US 75 near

Sherman, Texas. [NTSB09c] As the motorcoach approached Post Oak Creek its right steer axle failed and the motorcoach struck a seven-inch high curb at about a four-degree impact angle which it overrode and then struck a steel bridge railing. The motorcoach struck the railing at about 44 mi/hr and then slid along the railing for about 120 feet until it penetrated the bridge railing and fell about eight feet onto the creek embankment below. Seventeen passengers were fatally injured, the driver was seriously injured and 38 passengers received minor to serious injuries in the crash.

Figure 30. Site of a motorcoach bus crash in Sherman, TX, 2008. [NTSB09c]

US-75 in the area of the crash had a traffic volume of about 47,000 vehicles/day in 2006 and commercial vehicles accounted for 16 percent of the total traffic volume. The bridge railing at the crash site, shown in Figure 30, was a 27-inch tall steel beam and post system side-mounted on an 18-inch wide, seven-inch tall curb adjacent to a 22-inch wide shoulder. The bridge railing was 279-ft long. The bridge railing was a Texas Type II railing which was originally designed in 1954 in accordance with the AASHTO Bridge Design Specifications in effect at the time. Apparently, this bridge railing had been struck previously in 2001 by a tractor-trailer truck. It had penetrated the bridge railing causing some damage to the railing anchorages in the deck.

Based on its height alone this bridge railing would be classified today as no more than a TL3 railing but it is likely that it was never crash tested so its impact performance is doubtful.

As a result of the Sherman, Texas motorcoach crash, NTSB issued three safety recommendations dealing with the design and warranting of bridge railings.

H-09-17: "*Establish, in conjunction with the American Association of State Highway and Transportation Officials, performance and selection guidelines for bridge owners to use to develop objective warrants for high-performance Test Level Four, Five, and Six bridge railings applicable to new construction and rehabilitation projects where railing replacement is determined to be appropriate*."

H-09-25: "*Work with the Federal Highway Administration to establish performance and selection guidelines for bridge owners to use to develop objective warrants for high-performance Test Level Four, Five, and Six bridge railings applicable to new construction and rehabilitation projects where railing replacement is determined to be appropriate, and include the guidelines in the Load and Resistance Factor Design (LRFD) Bridge Design Specifications*."

H-09-26: *Revise Section 13 of the Load and Resistance Factor Design (LRFD) Bridge Design Specifications to state that bridge owners shall develop objective warrants for the selection and use of high-performance Test Level Four, Five, and Six bridge railings applicable to new construction and rehabilitation projects where railing replacement is determined to be appropriate*.

National Transportation Safety Board Recommendations

The NTSB first made recommendations on the design and selection of bridge railings in 1977 in its recommendations H-77-12 through 14 as a result of the Martinez, California crash discussed earlier. In 1980, the NTSB issued SEE-80-5 to, in part, assess FHWA's efforts in implementing the H-77-12 through 14 recommendations. SEE-80-5 contained an additional recommendation (i.e., H-80-64) which stated:

H-80-64: "Establish mandatory performance standards, and associated test procedures to be used in determining compliance, for all traffic barriers constructed on Federal-aid roads after January 1, 1982. The performance standards should first address automobiles and should be expanded for heavier passenger vehicles and trucks as research is completed to provide needed information." [NTSB80]

The result of these recommendations was an era of vigorous re-design and crash testing of bridge railings by the FHWA. In the 10 year period following H-80-64, 74 bridge railings were crash tested and accepted including W-Beam bridge railing, Thrie Beam bridge railing, Metal Tube bridge railing, Vertical Concrete parapet, New Jersey Barrier bridge railing, Tall Wall type, F-Shape Concrete Barrier bridge railing and Timber bridge railings. The FHWA published three Technical Memoranda where the States were advised of the requirements to crash test bridge railings, provided a list of the bridge railing designs that had been crash tested, and provided a list of all FHWA accepted bridge railings. [FHWA86; FHWA90; FHWA97a] Based on the results of this decade of research, the FHWA and AASHTO developed bridge railing design and crash test criteria which appeared as the 1989 Guide Specification to Bridge

Railings which will be discussed in a later section.[AASHTO89] The recommendation was "closed" by the NTSB and considered superseded by AASHTO's adoption of the Guide Specification. [AASHTO89]

Unfortunately, while the FHWA's design and crash testing research undertaken in the 1980's and the AASHTO Guide Specification made great advances in the design and testing of bridge railings, the warranting or selection of which types of bridge railings are best suited to which conditions was still very subjective. Recently, the FHWA added "Identification of Potentially Deficient Systems" and "Bridge Rail Retrofits" to their website. This website suggests that bridge railings designed prior to 1964 may not meet current specifications and includes a list of items to evaluate (e.g., base plate connections, anchor bolts, material brittleness, welding details, and reinforcement development). Additionally, the FHWA suggests that open-faced railings present a snagging hazard and that curbs or walkways in front of the bridge railings also present hazards. Suggestions for retrofitting any outdated bridge railings are included and discussed (e.g., concrete retrofit is economical if the structure can carry the added load; W-beam/thrie-beam retrofits provide an inexpensive, short-term solution; and metal post and beam retrofits for structures with walkways).[FHWA10] This website does not address the techniques one may use to conduct the investigation, how one may prioritize the need to retrofit deficient bridge railings or the identification of deficient bridge rails built after 1964.

Crash Testing

Crash testing is the most direct means of assessing barrier impact performance. Containment capacity is a function of the strength and height of a barrier. If a barrier is not strong enough, an impacting vehicle can penetrate through it (see Figure 31). If a barrier is not tall enough, an impacting vehicle can override or roll over it (see Figure 32). Full-scale crash testing is typically used to verify containment capacity of a barrier for a selected test level but testing guidelines do not indicate what traffic, geometric and operational characteristics should be used to assess the risk of a particular crash type occurring.

Hundreds of crash tests have been performed in the past several decades, many involving bridge railing designs for heavy vehicles. The purpose of this section is not to list or catalog all the various types of bridge railings that have been crash tested but, rather, to point out the different performance and test levels that have been used over the years to evaluate crash test performance.

Figure 31. Tractor Trailer Truck Penetration of Concrete Median Barrier.


Figure 32. Single-Unit Truck Rolling Over Concrete Bridge Rail.

NCHRP Report 230

NCHRP Report 230 was published in 1981 to provide guidelines for performing and evaluating full-scale vehicle crash tests of a variety of safety appurtenances including bridge railings. [Michie81] Report 230 did not explicitly include a multiple performance level or test level approach although it did include a number of supplemental tests for heavier vehicles such as utility buses (i.e., school buses), small and large intercity buses, tractor trailer trucks and tanker trailer trucks. The so-called “minimum” crash test matrix included small, medium and large passenger cars. The supplemental tests recommended in Report 230 were developed in conjunction with another NCHRP project (i.e., NCHRP Report 239) which presented recommendations for a multiple service level approach to bridge railing design. The supplemental tests were intended to satisfy the recommendations of Report 239 with respect to both lower containment bridge railings used on lower volume, low speed bridges and higher containment bridge railings intended for use in situations where bus and truck impacts would be more likely and more serious.

Report 239 included four service levels and attempted to establish the service levels based on the capacity of the bridge railings. The Report 230 supplemental tests were the means used to determine the bridge railing capacity.

1989 AASHTO Guide Specification for Bridge Railings

The 1989 AASHTO Guide Specification for Bridge Railings (GSBR) recommended that all bridge railings should be evaluated in full-scale crash tests to verify that a given bridge rail design meets the desired impact performance criteria. [AASHTO89] Three bridge rail performance levels and associated crash tests were recommended to access the performance of the bridge railings. 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 1800-lb passenger car and a 5400-lb pickup truck - were common to all three performance levels. Test conditions associated with Performance Level 1 (PL1) included the 5,400-lb pickup truck impacting at a speed of 45 mph and an angle of 20 degrees. For PL2, the speed of the pickup truck test was increased to 60 mph and a test with an 18,000 lb single unit truck impacting the barrier at a speed of 50 mph and an angle of 15 degrees was added to the test matrix. The highest performance level, PL3, incorporated a test with a 50,000 lb van-type tractor trailer impacting the barrier at a speed of 50 mph and an angle of 15 degrees. While crash testing procedures for bridge rails would quickly be wrapped into NCHRP Report 350 in 1993, only four years after the AASHTO GSBR was published, the concept of multiple performance levels was introduced for the first time in an AASHTO guide by the GSBR and this concept was retained and even expanded in Report 350.

Prior to 1989, there were a few higher performance bridge railings available but the publication of the 1989 GSBR and the later inclusion of bridge railings in Report 350 has resulted in a large number of bridge railings being designed and crash tested. The AASHTO-ARTBA-AGC Task Force 13 on-line bridge railing guide today (i.e., 2013) shows 117 crash-tested bridge railings; 59 of which are essentially PL1 bridge railings; 48 PL2 bridge railings and ten PL3 bridge railings. Today there are many crash tested FHWA-accepted bridge railings with demonstrated performance across a wide spectrum of test levels that were not available in 1989.

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. [Noel81; Beason89]

In the first study, an instrumented concrete wall (shown in Figure 33) was designed to, for the first time, measure the magnitude and location of vehicle impact forces. [Noel81] The wall consisted of four 10-ft long concrete panels, each laterally supported by four load cells. Each of the 42-in tall, 24-in thick panels was also instrumented with an accelerometer to account for inertia effects. Surfaces in contact with the supporting foundation and adjacent panels were Teflon coated to minimize friction. Eight full-scale crash tests were conducted using various sizes of passenger cars and buses ranging from an 1,800-lb sedan to a 32,020-lb intercity bus. In the second such study, a new wall with a height of 90 inches was constructed using similar design details. [Beason89] Three full-scale vehicle crash tests were performed with tractor-trailer vehicles ranging in weight from 50,000 lbs to 80,000 lbs.

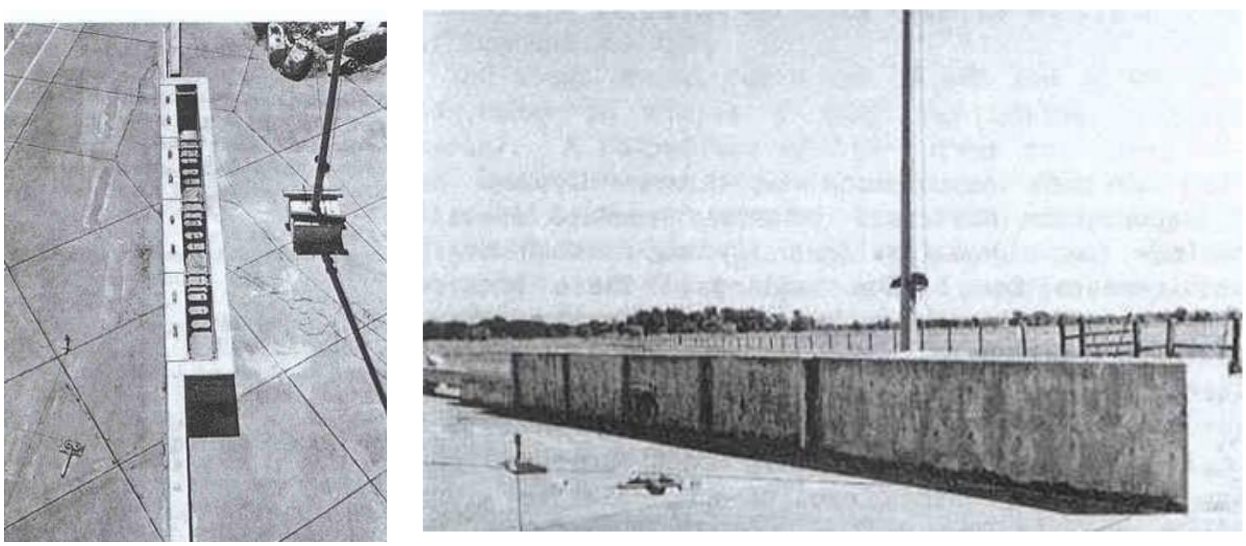


Figure 33. Instrumented crash wall. [Noel81]

The data from the instrumented wall tests were analyzed to determine the resultant magnitudes, locations, and distributions of the contact forces. Maximum forces were obtained by averaging the data over 50 msec intervals to reduce the effect of force “spikes” in the data that were believed to have little consequence to the required structural integrity of the bridge railings due to their short duration. The force measurements were obtained from a nearly rigid barrier and, therefore, were considered to represent the upper bound of forces that would be expected on an actual bridge railing. Any deformation of the bridge rail during impact would tend to reduce the magnitude of the impact forces below those obtained on the “nearly rigid” instrumented concrete wall.

NCHRP Report 350

In 1993, NCHRP Report 350 was published superseding the previous crash testing guidelines contained NCHRP Report 230. [Ross93] One major change in Report 350 is that six different test levels for roadside hardware were added for roadside hardware in general and bridge railings in particular. The intent was to provide test guidelines for developing a range of bridge railings that could be used in different situations. Test levels 1 through 3 are focused on the impact performance 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. The base test level for longitudinal barriers, including bridge railings, to be used on the National Highway System (NHS) is Test Level 3 (TL3). The structural adequacy test for this test level consists of a 2,000 kg (4,409 lb) pickup truck impacting a barrier at 100 km/h (62 mph) and 25 degrees.

Test levels 4 through 6 also include consideration of passenger vehicles, but additionally incorporate consideration of various sizes of trucks. Many state transportation departments require that their bridge railings meet Report 350 TL4. The impact conditions for NCHRP Report 350 TL4 involve an 8,000 kg (17,637 lb) single unit truck impacting the barrier at 80 km/h (50 mph) and 15 degrees. These impact conditions are similar to those associated with Performance Limit 2 (PL2) in the 1989 AASHTO “Guide Specification for Bridge Rails.” [AASHTO89]

A TL5 test involves an 80,000-lb (36,000-kg) van-type tractor trailer impacting the barrier at a speed of 50 mph (80 km/hr) and an angle of 15 degrees. Test Level 6 (TL6) uses the same impact conditions, but incorporates an 80,000-lb (36,000-kg) tractor-tank trailer. Barriers meeting these higher containment levels are used when the owner-agency considers that site conditions warrant the added expense. Site specific factors that might justify use of a high-containment barrier include a high percentage of heavy truck traffic or truck related crashes and/or an unusually high risk associated with barrier penetration. Such barriers are necessarily taller, stronger, and more expensive to construct. The higher test levels were intended for locations where there were a high percentage of trucks and where the consequences of trucks penetrating or rolling over the bridge railing would be severe. While Report 350 provided the testing recommendations, only general guidance was provided about what field conditions would indicate the need for a higher test level bridge railing.

FHWA has established approximate equivalences between the Report 239 multiple service levels, the three AASHTO GSBR performance levels and the Report 350 test levels in a memorandum to FHWA Regional Administrators in 1997. [Horne97] The equivalencies set out by the FHWA are summarized in Table 1, but the guidance from FHWA simply established crash test equivalencies without providing any guidance on when and where to use each type of bridge railing.

Table 1. Crash Test Acceptance Equivalencies from the FHWA. [Horne97]

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 (MASH)

Since the publication of NCHRP Report 350 in 1993, changes have occurred in vehicle fleet characteristics, operating conditions, technology, etc. 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 advancement and evolution of roadside safety testing and evaluation. The results of that research effort culminated in the 2009 AASHTO Manual for Assessing Safety Hardware (MASH) which superseded Report 350. [AASHTO09] MASH includes essentially the same test level approach with some changes to the impact conditions for the higher longitudinal barrier test levels. Barrier performance levels identified in MASH were modified from its predecessor, NCHRP Report 350. [AASHTO09; Ross93] These modifications were primarily related to the size of the test vehicles. Impact conditions associated six test levels tend to be calibrated off of impact conditions associated with Test Level 3 (TL3). TL3 is intended to represent barrier applications on typical high speed high-volume roadways since TL3 is the “default” test level used on the National Highway System. Impact speeds and angles for TL3 have traditionally been selected to be equal to the presumed 85th percentile impact speed and 85th percentile impact angle from ran off road crashes. Further, vehicle masses are normally selected to be equal to the 95th and 5th percentile values from the passenger car fleet. However, in recognition of the recent increase in the size of passenger vehicles and the expectation that high gasoline prices might push vehicle masses down, the light truck vehicle mass was reduced to the 90th percentile and the small car mass was reduced to the 2nd percentile of the 2002 new vehicle fleet.

Even with these adjustments, the severity of the TL3 test condition was increased significantly. 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. The primary concern was that if TL3 and TL4 converged, highway agencies would lose one of the longitudinal barrier options. The impact energy associated with the TL4 crash test conditions was increased by changing the single unit truck mass from 8,000 kg (17,637 lb) to 10,000 kg (22,000 lb) and the speed of the test vehicle from 80.47 km/h (50 mph) to 90.12 km/h (56 mph). This is particularly important to this study because the 57 percent increase in impact severity for MASH TL4 has resulted in higher design impact loads which will require stronger barriers and increased overturning moment which will require increased barrier height to prevent the heavier SUTs from rolling over the top of the barrier. In short, MASH TL4 barriers will likely have much higher capacities than Report 350 TL4 barriers.

While a barrier height of 32 inches satisfied NCHRP Report 350 TL4 impact conditions, recent crash testing conducted at the Midwest Roadside Safety Facility (MwRSF) at the University of Nebraska (UNL) under NCHRP Project 22-14(2) and the Texas Transportation Institute (TTI) under NCHRP Project 22-14(3) has demonstrated that taller barriers will be required to accommodate MASH TL4 (i.e., Report 350 and MASH TL4 are not equivalent). The Texas Department of Transportation (TxDOT) is currently sponsoring research to determine the minimum barrier height and design impact load for MASH TL4. [Sheikh11]

The vehicle specifications and impact conditions for TL5 and TL6 have not changed. Several user agencies have already begun applying the MASH criteria in their crash test programs. While no official crash test equivalencies have been released to compare Report 350 and MASH test level, Table 1 can be expanded to add the first line to represent the approximately equivalencies for MASH test levels.

Table 2. Approximate Crash Test Acceptance Equivalencies. [after Horne97]

<table>
<tr><th>Bridge Railing
Testing Criteria</th><th colspan="7">Acceptance Equivalencies</th></tr>
<tr><td>MASH</td><td>TL1</td><td colspan="2">TL2</td><td colspan="2">TL3</td><td>TL5</td><td>TL6</td></tr>
<tr><td>Report 350</td><td>TL1</td><td>TL2</td><td colspan="2">TL3</td><td>TL4</td><td>TL5</td><td>TL6</td></tr>
<tr><td>Report 230</td><td></td><td>MSL-1
MSL-2†</td><td colspan="2"></td><td></td><td></td><td></td></tr>
<tr><td>AASHTO Guide Spec</td><td></td><td>PL1</td><td colspan="2"></td><td>PL2</td><td>PL3</td><td></td></tr>
<tr><td>AASHTO LRFD Bridge Spec</td><td></td><td>PL1</td><td colspan="2"></td><td>PL2</td><td>PL3</td><td></td></tr>
</table>

† 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.

High Containment Barriers

Although fewer in number than TL3 and TL4 barriers, several barrier systems have been successfully designed and crash tested to TL5 criteria with an 80,000-lb tractor-van trailer. Only one barrier system is known to have been designed and successfully tested to TL6 with an 80,000-lb tractor-tank trailer (see Figure 34). [Hirsch85] TTI researchers conducted a study using a 90-in tall rigid instrumented concrete wall to quantify the magnitude and location of impact loads for a variety of trucks up to and including an 80,000 lb tractor trailer with both a van-type and tank-type trailer. [Beason89] Speeds in these tests ranged from 50 mph to 60 mph and the impact angles ranged from 15 degrees to 25 degrees.

Eleven additional tractor-trailer crash tests have been performed by TTI. In these tests, the gross vehicle weights ranged from 50,000 lbs to 80,000 lbs. Figure 35 shows a photograph from a test of a 42-in tall vertical wall bridge parapet being impacted by a 50,000 lb tractor-van trailer at a speed of 50 mph and an angle of 15 degrees. [Menges95] This test conforms to the impact conditions of Performance Level 3 (PL3) of the 1989 AASHTO "Guide Specification for Bridge Rails." Table 3 shows a summary of the test information and parameters for the tractor-trailer barrier impacts run at TTI.

Additionally, UNL researchers successfully developed both a 42-in. and a 51-in. tall, F-shape, half-section concrete barrier for TL5 impact criteria.[MWRSF10] For the 42-in. height, several configurations were provided with top barrier widths ranging from 10 in. to 12 in. and having barrier capacities ranging from 211 kips to 224 kips. Two preferred configurations were recommended for the 51-in. tall barrier having top barrier widths of 11 in. and 12 in. The size, quantity, and spacing of longitudinal and vertical steel reinforcing bars were selected to prevent concrete blowouts as well as prevent vehicle penetrations through or vaulting over the top of the barriers.

Figure 34. The only crash-tested TL6 bridge railing. [Hirsch85]

Figure 35. 50,000-lbs tractor trailer impacting a 42-inch vertical wall bridge railing. [Menges95]

Table 3. Summary of test information for selected heavy tractor-trailer crash tests.

Test date	Test No.	Barrier Type	Barrier Description	Van Trailer Type	Model Yr	Vehicle Make	Gross Weight (lb)	Impact Speed (mph)	Impact Angle (deg)	Pass/ Fail
1/16/2005	475680-1	Median Barrier	CA Type 60	36000V	1993	International	59,928	49.2		Pass
2/27/2004	475150-1	Median Barrier	CA Type 50 w/ Glare Screen	36000V	1989	Freightliner	45,236	41.3	34.0	Fail
12/12/1995	405511-02	Bridge Rail	1.07 m Vertical Wall	36000V	1983	Freightliner	79,200	49.8	14.5	Pass
8/9/1990	7162-01	Median Barrier	Ontario "Tall Wall"	80000A	1980	International	80,000	49.6	15.1	
7/11/1988	7069-13	Bridge Rail	42 in Vertical Wall	80000A	1979	International	50,050	51.4	16.2	Fail
5/27/1988	7046-09	Wall	Instrumented Wall	80000A	1979	International	50,000	50.4	14.6	
3/3/1988	7069-10	Bridge Rail	42 inch F-Shape	80000A	1979	International	50,000	52.2	14.0	Pass
5/8/1987	7046-04	Wall	Instrumented Wall	80000A	1971	Peterbilt	79,900	54.8	16.0	
4/7/1987	7046-03	Wall	Instrumented Wall	80000A	1973	White	80,080	55.0	15.3	
9/18/1984	2416-01	Bridge Rail	Mod. T5 w/ Metal Rail	80000A	1981	Kenworth	80,080	48.4	14.5	Pass
10/25/1983	2911-01	Bridge Rail	Modified T5	80000A	1980	Kenworth	80,120	51.4	15.0	Pass
5/26/1983	4798-13	Median Barrier	New Jersey Safety	80000A	1974	International	80,180	52.1	16.5	
7/14/1982	4348-02	Median Barrier	Modified Safety	80000A	1978	Autocar	80,420	52.8	16.0	
8/21/1981	2230-06	Bridge Rail	Modified C202	80000A	1978	AutoCar	79,770	49.1	15.0	Pass

Guidelines and Specifications

FHWA and AASHTO

AASHTO Standard Specifications for Highway Bridges

Historically, design of bridge rails has followed guidance contained in the AASHTO "Standard Specifications for Highway Bridges." Prior to 1965, the AASHTO specification required very simply that "substantial railings along each side of the bridge shall be provided for the protection of traffic." 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 in. 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.

These loads are only a fraction of what are used today. Based on poor accident history, accentuated by increased exposure due to dramatically increasing travel volumes, the engineering community came to realize that these criteria were inadequate. There was an urgent necessity for a railing specification that established loading requirements more in line with the weights and increased speeds of vehicles of that day.

Olson was perhaps the first to systematically examine the performance requirements for bridge railings in 1970. His results, documented in NCHRP Report 86, suggested using appropriate transitions, evaluation through crash testing, ability to minimize bridge rail penetration and many other things that are considered standard objectives of bridge railing design today. [Olson70] Bronstad built upon Olson's work in NCHRP Report 239 where he presented a multiple service level approach to selecting bridge railings. [Bronstad81] Bronstad's approach was a benefit-cost approach where the number of crashes exceeding the presumed capacity of the bridge railing was estimated. Four service levels were identified where the capacity was based on crash test results. Unfortunately, there was very little data available for Bronstad to use in developing his collision frequency and severity models so the resulting method was not definitive and was never widely adopted.

Revised bridge railing specifications were subsequently published in 1965 in the 9th edition of the AASHO "Standard Specifications for Highway Bridges." [AASHTO65] 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 as provided in an accompanying figure. Posts were designed for the transverse loading applied to each rail element plus a longitudinal load of half the transverse load. The transverse force on concrete parapet walls was distributed over a longitudinal length of 5 ft. 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.

These bridge rail design procedures were retained through numerous editions of the specifications. In fact, the provisions in the 17th edition of the AASHTO "Standard Specifications for Highway Bridges" published in 2002 are essentially the same as the specification adopted in 1965. [AASHTO02]

These requirements were intended to produce bridge rails that function adequately for passenger cars for a reasonable range of impact conditions. The reserve load capacity of the rail, beyond its elastic strength offers some factor of safety to accommodate more severe impact conditions or heavier vehicles. However, several catastrophic accidents involving large vehicles (e.g., buses and trucks) increased awareness of design requirements for bridge rails and the need to extend protection beyond passenger cars.

1989 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." [AASHTO89] The GSBR introduced several new and important concepts to the practice of selecting bridge railings including:

- Bridge railing performance should be demonstrated in full-scale crash tests.
- There should be multiple performance levels (i.e., three were recommended) to address the different risks and costs associated with specific traffic and bridge characteristics.
- A cost-benefit encroachment modeling software program, BCAP, was introduced to help designers make bridge railing selection decisions.
- Generic selection guidelines were presented that recommended the appropriate test level based on traffic volume, percent trucks, speed limit, horizontal curvature and grade.

The crash testing aspects of the 1989 AASHTO GSBR were discussed previously in the section on Crash Testing so this section will focus on the selection guidelines in the 1989 GSBR.

The 1989 AASHTO GSBR provided bridge engineers guidance for determining the appropriate railing performance level for a given bridge site. Selection guidelines were provided that estimated the appropriate railing performance level for a given bridge site based on highway and site characteristics. The highway and site characteristics used in the selection guidelines included 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 feet 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.

Table 4 shows a portion of the 1989 GSBR Table G2.7.1.3B which illustrates the selection recommendations. For example, if the design speed of a four-lane divided highway is 60 mi/hr (100 km/hr) and the percent of trucks is about 15 percent and the bridge railing is offset from the edge of travel eight feet, a PL1 railing is appropriate for traffic volumes up to 3,700 vpd; a PL2 bridge railing is recommended for traffic volumes between 3,700 and 31,900 vpd and a PL3 bridge railing is recommended for traffic volumes greater than 31,900 vpd. These recommendations presume that the highway section is straight with no grade, a deck height above the surface of 35 ft or less and the bridge does not pass over a sensitive or occupied area. If it does have horizontal curvature, grade or passes over a sensitive or occupied area, adjustment factors are presented. These adjustments are multiplied by the traffic volume in Table G2.7.1.3B.

Table 4. 60 mi/hr portion of the 1989 GSRB selection table.[AASHTO89]

Site Characteristics			Adjusted ADT Ranges for Bridge Railing Performance Levels (10^3 vpd)								
			Highway Type								
			Divided (or Undivided with 5 or more Lanes)			Undivided with 4 Lanes or Less			One Way		
DESIGN SPEED	PERCENT TRUCKS	BRIDGE RAIL OFFSET	PERFORMANCE LEVEL PL-1	PL-2	PL-3	PERFORMANCE LEVEL PL-1	PL-2	PL-3	PERFORMANCE LEVEL PL-1	PL-2	PL-3
60	0	0– 3	0 to	3.2 to	∞	0 to	2.0 to	∞	0 to	1.6 to	∞
60	0	3– 7	0 to	3.6 to	∞	0 to	2.3 to	∞	0 to	1.8 to	∞
60	0	7–12	0 to	4.4 to	∞	0 to	2.9 to	∞	0 to	2.2 to	∞
60	0	<12	0 to	5.5 to	∞	0 to	3.5 to	∞	0 to	2.8 to	∞
60	5	0– 3	0 to	3.0 to	107.3 to ∞	0 to	1.9 to	70.3 to ∞	0 to	1.5 to	53.7 to ∞
60	5	3– 7	0 to	3.3 to	126.3 to ∞	0 to	2.1 to	82.8 to ∞	0 to	1.7 to	63.2 to ∞
60	5	7–12	0 to	4.1 to	158.4 to ∞	0 to	2.7 to	105.6 to ∞	0 to	2.1 to	79.2 to ∞
60	5	>12	0 to	5.0 to	203.8 to ∞	0 to	3.3 to	138.2 to ∞	0 to	2.5 to	101.9 to ∞
60	10	0– 3	0 to	2.8 to	39.6 to ∞	0 to	1.8 to	25.0 to ∞	0 to	1.4 to	19.8 to ∞
60	10	3– 7	0 to	3.1 to	47.5 to ∞	0 to	2.0 to	29.3 to ∞	0 to	1.6 to	23.8 to ∞
60	10	7–12	0 to	3.9 to	53.1 to ∞	0 to	2.5 to	33.7 to ∞	0 to	2.0 to	26.6 to ∞
60	10	>12	0 to	4.7 to	67.6 to ∞	0 to	3.1 to	44.1 to ∞	0 to	2.4 to	33.8 to ∞
60	15	0– 3	0 to	2.7 to	24.3 to ∞	0 to	1.7 to	15.2 to ∞	0 to	1.4 to	12.2 to ∞
60	15	3– 7	0 to	2.9 to	29.3 to ∞	0 to	1.9 to	17.8 to ∞	0 to	1.5 to	14.7 to ∞
60	15	7–12	0 to	3.7 to	31.9 to ∞	0 to	2.4 to	20.0 to ∞	0 to	1.9 to	16.0 to ∞
60	15	>12	0 to	4.5 to	40.5 to ∞	0 to	2.9 to	26.2 to ∞	0 to	2.3 to	20.3 to ∞
60	20	0– 3	0 to	2.5 to	17.5 to ∞	0 to	1.6 to	10.9 to ∞	0 to	1.3 to	8.8 to ∞
60	20	3– 7	0 to	2.8 to	21.1 to ∞	0 to	1.8 to	12.8 to ∞	0 to	1.4 to	10.6 to ∞
60	20	7–12	0 to	3.5 to	22.8 to ∞	0 to	2.2 to	14.3 to ∞	0 to	1.8 to	11.4 to ∞
60	20	>12	0 to	4.2 to	28.9 to ∞	0 to	2.8 to	18.7 to ∞	0 to	2.1 to	14.5 to ∞
60	25	0– 3	0 to	2.4 to	13.7 to ∞	0 to	1.5 to	8.5 to ∞	0 to	1.2 to	6.9 to ∞
60	25	3– 7	0 to	2.6 to	16.5 to ∞	0 to	1.7 to	10.0 to ∞	0 to	1.3 to	8.3 to ∞
60	25	7–12	0 to	3.3 to	17.7 to ∞	0 to	2.1 to	11.1 to ∞	0 to	1.7 to	8.9 to ∞
60	25	>12	0 to	4.0 to	22.5 to ∞	0 to	2.6 to	14.5 to ∞	0 to	2.0 to	11.3 to ∞
60	30	0– 3	0 to	2.3 to	11.2 to ∞	0 to	1.4 to	7.0 to ∞	0 to	1.2 to	5.6 to ∞
60	30	3– 7	0 to	2.5 to	13.6 to ∞	0 to	1.6 to	8.2 to ∞	0 to	1.3 to	6.8 to ∞
60	30	7–12	0 to	3.2 to	14.5 to ∞	0 to	2.0 to	9.0 to ∞	0 to	1.6 to	7.3 to ∞
60	30	>12	0 to	3.8 to	18.4 to ∞	0 to	2.5 to	11.9 to ∞	0 to	1.9 to	9.2 to ∞
60	35	0– 3	0 to	2.2 to	9.5 to ∞	0 to	1.4 to	5.9 to ∞	0 to	1.1 to	4.8 to ∞
60	35	3– 7	0 to	2.4 to	11.5 to ∞	0 to	1.5 to	6.9 to ∞	0 to	1.2 to	5.8 to ∞
60	35	7–12	0 to	3.0 to	12.3 to ∞	0 to	1.9 to	7.7 to ∞	0 to	1.5 to	6.2 to ∞
60	35	>12	0 to	3.6 to	15.6 to ∞	0 to	2.4 to	10.0 to ∞	0 to	1.8 to	7.8 to ∞
60	40	0– 3	0 to	2.1 to	8.3 to ∞	0 to	1.3 to	5.1 to ∞	0 to	1.1 to	4.2 to ∞
60	40	3– 7	0 to	2.3 to	10.0 to ∞	0 to	1.4 to	6.0 to ∞	0 to	1.2 to	5.0 to ∞
60	40	7–12	0 to	2.9 to	10.6 to ∞	0 to	1.9 to	6.6 to ∞	0 to	1.5 to	5.3 to ∞
60	40	>12	0 to	3.5 to	13.5 to ∞	0 to	2.3 to	8.7 to ∞	0 to	1.8 to	6.8 to ∞

Returning to the example, if the highway being considered has an ADT of 28,000 vpd, 60 mi/hr design speed, 15 percent trucks and an 8-ft shoulder; Table 4 suggests a PL2 bridge railing is appropriate. If we assume that the bridge is on a vertical down grade of minus six percent, a horizontal curve of six degrees and is actually 50 ft above a high-occupancy land use surface below the adjustment factors would be 2.0, 2.0 and 1.8. The ADT would be adjust to 18,000 · 2.0 · 2.0 · 1.8 = 129,600 vpd which would place the railing into the PL3 category. The tables, therefore, allow the designer to select an appropriate rail based on the traffic volume, design speed, percent trucks, railing offset, horizontal curvature, grade, height of the structure and land use.

These selection procedures were developed using a benefit-cost analysis combined with engineering judgment. The benefit-cost analysis program (BCAP) estimated roadside encroachments, the consequences of these encroachments, and the cost of the consequences. An incremental benefit-cost ratio was computed to facilitate comparison of the relative merits or benefits of one design alternative to another. Table G2.7.1.3B was to be used for bridge railing selection unless the designer used the BCAP program.

The BCAP program will be discussed in more detail in a later section but obviously since Table G2.7.1.3B was based on the predictions of BCAP, the accuracy and validity of BCAP were fundamental to the validity of the recommendations. NCHRP Project 22-08 was initiated in order to assess BCAP and validate the 1989 AASHTO GSBR recommendations. [Mak94] Unfortunately, Mak and Sicking, the principal investigators for NCHRP 22-08, found some serious short comings of BCAP itself and the assumptions that were built into the selection tables. Mak and Sicking found that BCAP seriously over predicted bridge railing penetrations and seriously under predicted rollovers; the opposite of what would normally be expected. Based on crash test experience and anecdotal information, most bridge railings "fail" due to a heavy vehicle rolling over the barrier rather than penetrating after a structural failure so the BCAP results were counter intuitive. When a series of base-line simulations were performed with BCAP mimicking the GSBR recommendations, the researchers found that BCAP predicted 32.7 percent of tractor-trailer trucks striking a PL2 bridge railing would penetrate the bridge railing yet there were no predictions of rollover even though the center of gravity of a typical tractor trailer truck is 64 inches high and the typical PL2 barrier height was 32 inches high (i.e., the c.g. of the vehicle is 32 inches higher than the top of the barrier). [Mak94] Mak and Sicking discovered several reasons for this. One reason was the algorithm used to predict rollovers resulted in unreasonably high critical velocities. A new rollover algorithm was proposed and implemented as will be discussed in the later section devoted to BCAP.

Another reason involved barrier capacity. BCAP estimates the forces on the barrier using an algorithm first developed by Olson. The algorithm, as will be described later, is a simple derivation of the force based on the overall mechanics of the impact. After the impact force imparted by the vehicle is calculated, it is compared to the assumed bridge rail capacity. If the impact force is greater than the capacity, the bridge rail is considered failed. Estimating the actual capacity of bridge railings is more difficult than it might first seem. Materials are routinely assumed to be less strong and loads are routinely over estimated in design so even if the theoretical capacity is calculated it is likely a very conservative value. For example, in designing concrete structures a resistance factor 0.85 is usually used for bending which essentially takes advantage of only 85 percent of the strength of concrete. Likewise, if an allowable stress design method for steel were used, 67 percent of the strength of the steel is assumed. In both cases, the designer is neglecting a significant portion of the capacity of the structure. While this makes excellent design sense, it makes it difficult to estimate the real failure conditions of the structure. BCAP assumed that PL1 bridge railings have a capacity of 15 kips, PL2 railings have 35 kips and PL3 railings have 55 kips.   While there are relatively few crash tests where structural failure of the bridge railing was observed, Mak and Sicking were able to find some cases where the bridge railing experienced some degree of structural failure (i.e., hairline cracking, spalling, etc.). When they compared the limited crash test results to the BCAP assumptions they found that the BCAP assumptions were about half what could be supported by crash tests as shown in Table 5.

Table 5. Bridge railing capacity recommendations in BCAP and NCHRP 22-08.[after Mak94]

Performance Level	BCAP Assumption (kips)	Mak/Sicking Recommendation (kips)
PL1	15	30
PL2	35	64
PL3	55	108

Adding to the difficulty is the basic assumption in BCAP that when capacity is reached, the bridge railing will totally fail and allow the vehicle to penetrate. In fact, this does not generally happen. Bridge railings can experience structural failure and sometimes will still redirect the vehicle. The “failure” may be cracks or spalls that are considered serious damage to the bridge rail, but the bridge rail may still have enough structural integrity to prevent penetration by the vehicle.

Recently, Alberson and others evaluated a 32-inch high PL2 concrete safety shaped barrier, shown earlier in Figure 6, that had experienced structural failure problems in the field.[Alberson11; Alberson04] A yield-line structural analysis was performed on the bridge railing which resulted in an estimate of the barrier capacity of 33.6 kips when loaded near a construction joint and 47.7 kips when loaded at the mid-span. The same design was then constructed and statically tested to failure resulting in a near-the-joint capacity of 35.1 kips and a mid-span capacity of 45.1 kips. The bridge railing was also subjected to full-scale Report 350 TL4 crash tests which were passed successfully and which caused relatively minor concrete damage (e.g., hairline cracks and some gouging). As shown by Alberson’s research, the capacity values suggested by the 1989 AASHTO GSBR were grossly over conservative and those proposed by Mak and Sicking were more appropriate although it should be noted that this particular railing was chosen for investigation precisely because there had been some observed field structural failures so this particular railing probably represents the lower end of the capacity of PL2 railings.

Since BCAP first assesses the capacity and then the rollover potential, the overly conservative values for capacity tended to predict too many penetrations. Since the higher velocity truck impacts would tend to reach the capacity too early and the rollover algorithm was under conservative, penetrations were over predicted and rollovers under predicted.

Mak and Sicking revised the rollover algorithm and adjusted the bridge railing capacities upward as shown in Table 5 and re-ran their analysis. For example, 32.7 percent of tractor trailer truck crashes penetrated the railing and none rolled over in the initial BCAP runs whereas after the improvements implemented by Mak and Sicking 3.4 percent penetrated which seemed more reasonable.

Mak and Sicking also evaluated bridge railing crash data from Texas as will be discussed in more detail in a later section. [Mak94] Mak and Sicking found that the Texas data indicated that 2.2 percent of bridge railing crashes result in the vehicle going through (i.e., penetration) or over (i.e., roll over the barrier) and they believed that even this value was a high-side estimate due to coding errors on the police crash reports. The improved BCAP with the higher capacity limits and improved rollover algorithm resulted in an overall estimate of 10 percent going through (i.e., 1.2 percent penetrating and 8.9 percent rolling over) for the typical Texas conditions so even the improved BCAP appeared to over predict penetrations/rollover by an order of magnitude although the proportion of penetrations to rollovers appears much more reasonable. In short, then, BCAP and the 1989 AASHTO GSBR appear to over predict bridge railing penetrations and under predict rollovers. The improvements from NCHRP 22-08 appeared to improve the results although even the improved BCAP over predicts the incidence of vehicles going through or over the bridge railing.

Mak and Sicking developed new versions of the selection tables based on the improved version of BCAP. These new selection tables were structured in an identical way to the prior tables but the traffic volume cutoffs were higher. Table 6 shows the portion of the revised recommendations from NCHRP Project 22-08 that corresponds to Table 4 shown earlier. In the example presented earlier, a PL2 bridge railing would be recommended for a four-lane divided highway with a 60 mi/hr (100 km/hr) design speed, 15 percent trucks and eight-foot offset from the travelled way and no adjustments for traffic volumes between 3,700 and 31,900 vpd whereas Table 6 would suggest a PL2 bridge railing under the same conditions is appropriate for traffic volumes of 17,000 to 51,000 vpd; much higher than the 1989 GSBR.

The final report for NCHRP 22-08 was never published since NCHRP Report 350 appeared about the same time and the RSAP program was also released. It was thought that bridge railing selection guidelines would work themselves out in the process of replacing the crash testing recommendations of the 1989 GSBR with the new recommendations of Report 350 and replacing BCAP with RSAP. Unfortunately, such was not the case and the 1989 GSBR was never up-dated and its recommendations were never superseded.

Table 6. Revised selection guidelines for bridge railings based on NCHRP 22-08. [Mak94]

Site Characteristics			Adjusted ADT Ranges for Bridge Railing Performance Levels (10^3 vpd)											
			Highway Type											
			Divided (or Undivided with 5 or more lanes)				Undivided with 4 Lanes or Less				One Way			
		Bridge	Performance Level				Performance Level				Performance Level			
Design Speed	Percent Trucks	Rail Offset	PL-1	PL-2	PL-3		PL-1	PL-2	PL-3		PL-1	PL-2	PL-3	
60	0	0- 3	0 to	60.0 to	∞		0 to	44.3 to	∞		0 to	30.0 to	∞	
60	0	3- 7	0 to	102.6 to	∞		0 to	73.2 to	∞		0 to	51.3 to	∞	
60	0	7-12	0 to	140.6 to	∞		0 to	98.6 to	∞		0 to	70.3 to	∞	
60	0	>12	0 to	177.8 to	∞		0 to	129.3 to	∞		0 to	88.9 to	∞	
60	5	0- 3	0 to	32.8 to	154.6 to	∞	0 to	22.4 to	99.5 to	∞	0 to	16.4 to	77.3 to	∞
60	5	3- 7	0 to	45.2 to	179.6 to	∞	0 to	29.8 to	117.7 to	∞	0 to	22.6 to	89.8 to	∞
60	5	7-12	0 to	60.2 to	206.8 to	∞	0 to	39.5 to	146.2 to	∞	0 to	30.1 to	103.4 to	∞
60	5	>12	0 to	73.0 to	295.0 to	∞	0 to	50.0 to	217.2 to	∞	0 to	36.5 to	147.5 to	∞
60	10	0- 3	0 to	18.0 to	63.6 to	∞	0 to	11.6 to	39.3 to	∞	0 to	9.0 to	31.8 to	∞
60	10	3- 7	0 to	22.8 to	71.2 to	∞	0 to	14.3 to	45.9 to	∞	0 to	11.4 to	35.6 to	∞
60	10	7-12	0 to	27.6 to	82.6 to	∞	0 to	18.0 to	57.3 to	∞	0 to	13.8 to	41.3 to	∞
60	10	>12	0 to	34.0 to	107.2 to	∞	0 to	22.8 to	79.1 to	∞	0 to	17.0 to	53.6 to	∞
60	15	0- 3	0 to	12.4 to	40.0 to	∞	0 to	7.9 to	24.5 to	∞	0 to	6.2 to	20.0 to	∞
60	15	3- 7	0 to	15.2 to	44.4 to	∞	0 to	9.4 to	28.5 to	∞	0 to	7.6 to	22.2 to	∞
60	15	7-12	0 to	17.8 to	51.6 to	∞	0 to	11.7 to	35.6 to	∞	0 to	8.9 to	25.8 to	∞
60	15	>12	0 to	22.2 to	65.6 to	∞	0 to	14.8 to	48.3 to	∞	0 to	11.1 to	32.8 to	∞
60	20	0- 3	0 to	9.4 to	29.2 to	∞	0 to	5.9 to	17.8 to	∞	0 to	4.7 to	14.6 to	∞
60	20	3- 7	0 to	11.4 to	32.2 to	∞	0 to	7.0 to	20.7 to	∞	0 to	5.7 to	16.1 to	∞
60	20	7-12	0 to	13.2 to	37.4 to	∞	0 to	8.6 to	25.8 to	∞	0 to	6.6 to	18.7 to	∞
60	20	>12	0 to	16.4 to	47.2 to	∞	0 to	10.9 to	34.8 to	∞	0 to	8.2 to	23.6 to	∞
60	25	0- 3	0 to	7.6 to	23.0 to	∞	0 to	4.8 to	13.9 to	∞	0 to	3.8 to	11.5 to	∞
60	25	3- 7	0 to	9.2 to	25.4 to	∞	0 to	5.6 to	16.2 to	∞	0 to	4.6 to	12.7 to	∞
60	25	7-12	0 to	10.4 to	29.4 to	∞	0 to	6.8 to	20.3 to	∞	0 to	5.2 to	14.7 to	∞
60	25	>12	0 to	13.2 to	36.8 to	∞	0 to	8.7 to	27.2 to	∞	0 to	6.6 to	18.4 to	∞
60	30	0- 3	0 to	6.4 to	19.0 to	∞	0 to	4.0 to	11.5 to	∞	0 to	3.2 to	9.5 to	∞
60	30	3- 7	0 to	7.6 to	20.8 to	∞	0 to	4.6 to	13.4 to	∞	0 to	3.8 to	10.4 to	∞
60	30	7-12	0 to	8.6 to	24.2 to	∞	0 to	5.7 to	16.7 to	∞	0 to	4.3 to	12.1 to	∞
60	30	>12	0 to	10.8 to	30.2 to	∞	0 to	7.2 to	22.3 to	∞	0 to	5.4 to	15.1 to	∞
60	35	0- 3	0 to	5.4 to	16.2 to	∞	0 to	3.4 to	9.7 to	∞	0 to	2.7 to	8.1 to	∞
60	35	3- 7	0 to	6.6 to	17.8 to	∞	0 to	4.0 to	11.3 to	∞	0 to	3.3 to	8.9 to	∞
60	35	7-12	0 to	7.4 to	20.6 to	∞	0 to	4.8 to	14.2 to	∞	0 to	3.7 to	10.3 to	∞
60	35	>12	0 to	9.2 to	25.6 to	∞	0 to	6.1 to	18.9 to	∞	0 to	4.6 to	12.8 to	∞
60	40	0- 3	0 to	4.8 to	14.0 to	∞	0 to	3.0 to	8.5 to	∞	0 to	2.4 to	7.0 to	∞
60	40	3- 7	0 to	5.8 to	15.4 to	∞	0 to	3.5 to	9.9 to	∞	0 to	2.9 to	7.7 to	∞
60	40	7-12	0 to	6.4 to	18.0 to	∞	0 to	4.2 to	12.3 to	∞	0 to	3.2 to	9.0 to	∞
60	40	>12	0 to	8.2 to	22.2 to	∞	0 to	5.4 to	16.4 to	∞	0 to	4.1 to	11.1 to	∞

Roadside Design Guide

1989 also was the year that AASHTO first published the Roadside Design Guide (RDG). The RDG is a comprehensive guide to designing many aspects of the roadside but Chapter 7 deals exclusively with bridge railings.[AASHTO89] The subject of test level selection procedures is addressed briefly in section 7.3 of the RDG but the reader is referred back to section 5.3 for general guidance on traffic and operational characteristics that should be used in selecting the appropriate test level barrier.

RDG section 5.3 lists the following three subjective criteria that should be used in choosing an appropriate test level barrier:

1. Percentage of heavy vehicles,
2. Adverse geometrics (e.g., small-radius horizontal curves),
3. Severe consequences of a penetration by a heavy vehicle.

In essence, the RDG restates the generally philosophy of the 1989 GSBR without providing any additional specific information.

2004 AASHTO A Policy on the Geometric Design of Highways and Streets

The AASHTO document "A Policy on the Geometric Design of Highways and Streets" (i.e., the Green Book) discusses the subject of bridge railings while addressing interchanges, underpasses, and overpasses. Specifically, the Green Book recommends that "the design vehicle should be safely redirected, without penetration or vaulting over the railing … the railings should not pocket or snag the design vehicle, causing abrupt deceleration or spinout; and it should not cause the design vehicle to roll over." [AASHTO04]

Bridge railings may limit the sight distance at interchanges, intersections, on ramps and along the road. The Green Book acknowledges this concern and suggests the bridge railing "should provide a freedom of view … insofar as practical; however, capability to redirect errant vehicles should have precedence over preserving the motorist's view." [AASHTO04] Adjustments to the horizontal alignment are suggested to improve sight distance, when feasible.

When pedestrians or bicycles are accommodated on the bridge, the Green Book suggests "a barrier-type bridge rail of adequate height should be installed between the pedestrian walkway and the roadway. Also, a pedestrian rail or screen should be provided on the outer edge of the walkway." [AASHTO04]

FHWA Supplemental Guidance on Accommodating Heavy Vehicles on US Highways

The FHWA Office of Safety issued supplemental guidance on accommodating heavy vehicles on US Highways in a 2004 report. [FHWA04] The report addresses both geometric design barrier design, and placement issues. According to the report, there were 302 fatal single vehicle truck crashes in 2002 involving van, cargo, flat-bed or dump type trailers. Of these 302 fatal crashes, the first harmful event in 26 (8.6 percent) of these cases was listed as guardrail, concrete traffic barrier or bridge railing.

The FHWA report goes on to note that there are no specific warrants for the use of higher performance or test level barriers because heavy vehicle impacts are generally rare events. The report repeats the general guidance found in the RDG for subjective factors including (1) a high percentage of trucks, (2) adverse geometrics and/or poor sight distance and (3) potentially severe consequences associated with the truck penetrating the barrier and offers some additional specific guidance [FHWA04].

This 2004 FHWA report cites a 1997 FHWA policy memorandum which formally adopted NCHRP Report 350 as the guideline for testing bridge railings. This 1997 memorandum summarized over a decade of crash tests conducted on bridge railings, providing a complete list of all the crash tested bridge railings, the guidelines used to test the bridge railings (e.g., NCHRP 230, NCHRP 350, etc.), "equivalency" listings for other test standards, and sketches for construction of the railings. This memorandum also established that the minimum acceptable bridge railing acceptable on the national network will be Report 350 TL3, "unless supported by a rational selection procedure." The States were not, as a result of this memorandum, required to upgrade existing bridge railings, beyond normal improvements. [FHWA97b]

The FHWA issued another policy memorandum in 2010, "Design Considerations for Prevention of Cargo Tank Rollovers" in response to another fatal tanker truck crash investigated by NTSB. [FHWA10] This memorandum reiterated the guidance in the 2004 FHWA report for accommodating heavy vehicles, while adding some additional guidance. Some geometric factors the FHWA suggests when selecting a bridge railing are:

- Conflict points,
- Dramatic horizontal and/or vertical alignments,
- Lowering of the design speed, and
- Super-elevation which may increase large vehicle instability.

Some highway characteristics the FHWA suggests considering in the selection of bridge railings:

- High volume highways or other such facilities (i.e., transit, commuter rail, etc.) located beneath a bridge,
- Facilities where an impact could lead to catastrophic loss of life (i.e., chemical plants, nuclear facilities, etc.),
- Sensitive environmental areas (i.e., public water supplies), or
- Regionally or nationally significant bridges and tunnels.

In summary, several decades of recommendations by NTSB, crash testing by FHWA, and multiple national research projects have resulted in general guidance from the FHWA for the selection of bridge rails based on geometric factors and highway characteristics but there is still relatively little specific guidance on the selection of bridge railings to fit specific local conditions.

The States

Many states refer designers to Chapter 13 of the AASHTO LRFD Bridge Design Specifications for strength and geometric requirements, NCHRP 350 for crash test criteria, the 1989 AASHTO Guide Specification for warrants based on ADT, design speed, percentage truck traffic and horizontal and vertical geometry while noting that there is ongoing research to evaluate the warrants in the 1989 Specification. These national documents are supplemented in many States with a Bridge or Structures Design Manual in which the States detail the use of specific railings under certain situations, specify particular test levels for certain roadways and discuss retrofit polices. The Bridge or Structures Design Manuals from many different States have been reviewed and are discussed in this section.

Florida DOT

The Florida Department of Transportation's Structures Design Guidelines provide guidance to designers on the selection of bridge railings. [FDOT11] This document provides the following guidance for the installation of bridge railings, which extends to all construction, including new, temporary, 3R (e.g., Resurfacing, Restoration and Rehabilitation) and widening projects:

- Permanent installations must install a successfully crash tested TL4, TL5 or TL6 bridge railing.
- Temporary installations must install a successfully crash tested TL3 (minimum) when shielding drop-offs. TL2 (minimum) may be used when shielding work zones without drop-offs and a design speed of 45 mph or less.
- Upgrade both sides of a structure "when widening work is proposed for only one side and the existing traffic railing on the non-widened side does not meet the criteria for new traffic railings." [FDOT11]

Designers should provide a TL5 or TL6 bridge railing "when any of the following conditions exist:

- The volume of truck traffic is unusually high.
- A vehicle penetrating or overtopping the traffic railing would cause high risk to the public or surrounding facilities.
- The alignment is sharply curved with moderate to heavy truck traffic."[FDOT11]

Standard bridge railing designs are suggested by FDOT, however, the use of non-FDOT standard railings is permitted provided the railings meet the requirements listed above and following the review and approval of the FDOT Structures Design Office.

When rehabilitation or renovation work is proposed on an existing structure and the bridge railing does not meet the criteria detailed above, the existing railing should be replaced or retrofitted to meet the TL4 minimum performance standards. When selecting a replacement or retrofit bridge railing, FDOT suggests that designers evaluate the following aspects of the project:

1. "Elements of the structure.
    - Width, alignment and grade of roadway along structure.
    - Type, aesthetics, and strength of existing railing.
    - Structure length.
    - Potential for posting speed limits in the vicinity of the structure.
    - Potential for establishing no-passing zones in the vicinity of the structure.

- Approach and trailing end treatments (guardrail, crash cushion or rigid shoulder barrier).
- Strength of supporting bridge deck or wall.
- Load rating of existing bridge.

2. Characteristics of the structure location.
   - Position of adjacent streets and their average daily traffic.
   - Structure height above lower terrain or waterway.
   - Approach roadways width, alignment and grade.
   - Design speed, posted speed, average daily traffic and percentage of truck traffic.
   - Accident history on the structure.
   - Traffic control required for initial construction of retrofit and for potential future repairs.
   - Locations and characteristics of pedestrian facilities / features (if present).
3. Features of the retrofit designs.
   - Placement or spacing of anchor bolts or dowels.
   - Reinforcement anchorage and potential conflicts with existing reinforcement, voids, conduits, etc.
   - Self-weight of retrofit railing.
   - End treatments.
   - Effects on pedestrian facilities.
   - Evaluation of existing supporting structure strength for traffic railing retrofits."[FDOT11]

FDOT suggests the use of the modified thrie beam guardrail or vertical face traffic railing retrofits which are based on successfully crash-tested TL4 designs. Modifications to the designs are offered to designers which should work with the various existing Florida bridges.[FDOT11]

Minnesota DOT

The Minnesota Department of Transportation publishes the Minnesota Bridge Design Manual where bridge railing application is discussed.[MNDOT06] The Manual requires that "railing designs shall include consideration of safety, cost, aesthetics and maintenance." [MNDOT06] The Bridge Design Manual details the use of TL2 through TL5 bridge railings for uses with sidewalks, bicycles, various design speeds, and different geometrics. Different bridge railings are also specified for specific routes. These specifications have been summarized and are shown in Table 7.

Table 7 Summary of Minnesota "TABLE 13.2.1: Standard Rail Applications" [MNDOT06]

Description	Test Level	Speed Limit	Application	Comment
Concrete Barrier (Type P-1) and Metal Railing	TL2	≤ 40 mph	Outside edge of walk on highway bridges with sidewalks where bicycle traffic on the walk is expected and protective screening is not required.	2'-4" parapet with 2'-2" metal rail
Cloquet Bridge Railing Bridge No. 09008 and 09009				2'-2 3/4" metal rail on 2'-4" parapet
Concrete Barrier (Type P-1) and Wire Fence	TL2	≤ 40 mph	Highway bridges with walks. This rail is used on the outside edge of walk and meets bicycle and protective screening requirements.	2'-4" parapet and 6' metal rail with chain link fabric.
Concrete Barrier (Type P-1) and Tube Railing with Fence				2'-4" parapet and 5'-8 ½" metal rail with chain link fabric
Concrete Barrier (Type P-3) Ornamental				3'-9" metal rail on 2'-4" parapet
St. Peter Railing				4'-6" metal rail on 2'-4" parapet
TH 100 Corridor Standard				3'-9" metal rail on 2'-4" parapet
TH 212 Corridor Standard				5'-8' to 9'-2" metal rail on 2'-4" parapet
TH 610 Corridor Standard				5'-51/2" metal rail on 2'-4"parapet
Victoria Street Railing				5'-8" metal rail on 2'-4" parapet
Concrete Barrier (Type F or P-4)	TL4	All	Traffic Only	2'-8" tall
Concrete Barrier (Type P-2) and Structural Tube Railing	TL4	All	Traffic Only, where an aesthetic railing is desired.	1'-3" metal railing on 1'-9" parapet
Concrete Barrier (Type F)	TL5	> 40 mph	High Protection Area where Dc > 5° and Speed > 40 mph.	3'-6" tall
Concrete Barrier (Type F)	TL5	All	Between sidewalk and roadway where the shoulder is < 6'.	
Concrete Barrier (Type F)	TL5	All	Bridges with designated bike path or where glare screen is required.	4'-8" tall (The additional height is to protect a bicycle rider.)

New York State DOT

The New York State DOT uses the following recommendations for each performance level, as outlined in the New York State DOT Bridge Manual: [NYSDOT10]

- "TL2 (PL1)–Taken to be generally acceptable for most local and collector roads with favorable site conditions, work zones and where a small number of heavy vehicles are expected and posted speeds are reduced.
- TL4 (PL2)–Taken to be generally acceptable for the majority of applications on high-speed highways, expressways and interstate highways with a mixture of trucks and heavy vehicles.
- TL5 (PL3)–Taken to be generally acceptable for applications on high-speed, high-traffic volume and high ratio of heavy vehicles for expressways and interstate highways with unfavorable site conditions."

The railing functional and geometric characteristics are considered. These criteria include the under-crossing features, pedestrian accommodations, and bicycle accommodations. The formal accommodation of pedestrian and bicycle traffic requires the use of 3.5 foot tall railings.

Table 6-1 of the NYSDOT Bridge Manual outlines bridge railing designs by various AADTs for TL2 through TL5 which are appropriate for various scenarios including pedestrian and bicycle accommodations and different under-crossing features.

Interstate and other controlled-access, high-speed highways are mandated to have concrete bridge railings although parkways without truck traffic and culvert structures are excluded. Steel railings are currently not permitted on interstates or for other TL5 uses since, as the document in-correctly states, there were "no known steel railing systems" crash tested for TL5.

In fact, concrete is the first choice of material for most of the bridge railing design categories described above. "This preference is based on the concrete barrier's strength, durability and low initial and maintenance costs compared to metal railing systems. Factors that may cause an alternative selection to be made are:"[NYSDOT10]

- Bridge Deck Drainage
- Aesthetics
- Visibility
- Snow Accumulation

"Railing treatments on rehabilitation projects is a complex subject with many project specific considerations….engineering judgment will be required." Safety is the first considered when deciding on whether or not to upgrade bridge railing on a rehabilitation project. After determining the long-term planning strategy for the area and the structure, the existing railings are examined to determine if the railings meet NCHRP 230 crash test criteria. If the railing does meet those criteria, the existing railings are often retrofitted with guardrail if there is a safety walk or allowed to remain in place as is. If the bridge railings do not meet the NCHRP230 or later criteria, the railings are upgraded. [NYSDOT10]

New Jersey DOT

The New Jersey DOT specifies, through the New Jersey Design Manual for Bridges and Structures, 5th Edition that TL5 systems are considered the minimally accepted system for bridges which carry interstate traffic. [NJDOT10] TL4 systems are used for other NHS classified roadways. On non-NHS/non-State owned roadways, the use of TL1, TL2, and TL3 bridge railings are permitted. NJDOT suggests that the use of crash test specifications outlined in NCHRP 350 be used to determine a test level which best corresponds with the roadway when choosing a test level less than TL4, specifically considering design speed, truck traffic and how the roadway characteristics relate to the test level specifications to determine an appropriate test level for non-NHS roadways.

There are two TL5 systems used on interstate bridges in New Jersey: the 42-inch tall “F” shape concrete barrier is recommended for heavy vehicle containment and for horizontal curves of less than 1000 feet or on exit ramps and the 50-inch high Texas Type HT railing is used in conjunction with noise barriers where heavy vehicle over-tipping and potentially damaging the noise barrier is a concern.[NJDOT10]

Rhode Island DOT

The Rhode Island LRFD Bridge Design Manual states that “all railings systems shall meet the full-scale crash-test criteria as established in the NCHRP Report 350.” [RIDOT11] For new construction, TL4 bridge railings are the minimum test level barrier installed, except on interstate highways where TL5 bridge railing shall be installed.

TL5 bridge railings are also considered when the bridge is expected to experience high traffic volumes, high speeds with high truck percentages, or unfavorable site conditions. Unfavorable conditions may include:

- “High occupancy land uses below the bridge,
- Deep water below the bridge,
- Steep profile grades on or approaching the bridge,
- High curvature along the alignment of the bridge,
- Anticipated excessive number of van-type tractor trailers, or
- Any other set of conditions which, through sound engineering judgment, may justify a higher level of railing resistance.” [RIDOT11]

The installation of barriers with a test level less than TL4 may be considered when the ADT is less than 500 vehicles per day, the percentage of trucks is less than or equal to 5%, the design speed is less than 40 mph, the bridge is on a tangent section *and* the bridge deck height is less than 28 feet above ground or water surface elevation. Minor detail changes to existing, crash-tested railing systems are permitted, provided "engineering judgment and/or analysis" is used to determine the need for additional crash-testing. [RIDOT11]

Michigan DOT

In contrast to some other states, Michigan does not specify the minimum test level for bridge railings but it does require upgrading all railings when the bridge deck is replaced. [MDOT09] Regarding the installation of bridge railing, the MDOT Bridge Design Manual states simply that the railing "shall be of a type that has passed full scale impact (crash) tests" and provides a reference to five standard MDOT railings:

- Type 4 Barrier - Standard Plan B-17-Series,
- Type 5 Barrier - Standard Plan B-20-Series,
- 2 Tube railing - Standard PlanB-21-Series,
- 4 Tube railing – Standard Plan B-26-series and
- Aesthetic Parapet Tube railing - Standard Plan B-25-Series.[MDOT09]

Massachusetts DOT

The Massachusetts Department of Transportation specifies the use of TL2, TL4 and TL5 barriers under different situations in the MassHighway Bridge Manual.[MassDOT10] The following circumstances dictate the use of these different test levels:

- TL2 bridge railings may only be used on non-NHS roadways with speeds not exceeding 45mph.
- TL4 bridge railings may be used on NHS and Non-NHS highways, except limited access highways and their ramps
- TL5 bridge railings must be used on limited access highways and the ramps. This includes interstate highways, NHS and Non-NHS highways.

Details for bridge railing designs for use in specific situations (e.g.., where pedestrian are permitted or forbidden, bridges over electrified rail road tracks, municipally owned bridges, etc.) are provided. Bridge railings other than the standard railings are permitted provided that the use of non-standard railings "can be justified and that they have been crash tested."[MassDOT10]

Ohio DOT

An Ohio DOT inter-office memorandum restates the 2003 Design Manual policy which established the minimum acceptable bridge railing shall be TL3, however, now Ohio will allow the existing TL2 Deep Beam Bridge Railings to be maintained provided they are in good condition. [ODOT02]

North Carolina DOT

The North Carolina Department of Transportation supplements national standards with its Structures Design Manual where a minimum bridge railing of TL3 is established. [NCDOT10] TL2 or aesthetic railings are permitted under the following situations:

- "Non-NHS routes,
- Design speeds less than or equal to 45 mph, or
- In conjunction with a sidewalk." [NCDOT10]

NCDOT suggests the use of vertical concrete barrier rail for bridges on NHS and non-NHS routes. Bridges which accommodate pedestrians can add height to the railing through an added metal railing. When conducting an overlay on the bridge, a minimum rail height is established and noted on the plans to be maintained during construction, however, guidance on establishing the minimum height is not provided in the Structures Design Manual.[NCDOT10]

North Dakota DOT

North Dakota publishes a Design Manual which includes specifications for all facets of highway design within the state of North Dakota. [NDDOT09] The bridge chapter has specifications for the installation of new and retrofit bridge railings based on height and refers designers to AASHTO for the latest specifications. NDDOT requires all new bridge railing to be 32 inches tall (i.e., essentially Report 350 TL4) while existing railing may be retrofitted with two-tubes where applicable. Bridge railings on sidewalks are to be a minimum of 42 inches tall on the outer edge. Bridge railings on shared-use paths (i.e., bicycle and pedestrian) are to be a minimum of 54 inches tall on the outer edge. Both are to be crash tested. [NDDOT09]

Illinois DOT

The Illinois DOT provides guidance on the selection of the appropriate bridge railing test level in its Bridge Manual where IDOT states that the owner of a structure is "responsible for determining the test level necessary for each application" and that railings on all new or rehabilitated bridges on Federal and State routes shall be TL4. [IDOT09] The preferred bridge railing is the 34-inch TL4 F-Shape. A 42-inch high TL5 F-shape bridge railing "should only be used in the following scenarios:

1. Structures with a future DHV (one way) × % trucks greater than 250.
2. Structures located in areas with high incidences of truck rollover accidents.
3. Structures with a radius of 1000 ft. or less with truck traffic."[IDOT09]

Following these guidelines, structures carrying 10% or more truck traffic and an ADT of 5,000 vpd or more should install a TL5 bridge railing.

Indiana DOT

Indiana DOT allows the use of bridge railing designs ranging in performance from TL2 through TL5 but does not include specifications for the specific installation of any particular test level under any particular situation. Designers are referred to the AASHTO LRFD Bridge Design Specifications.[INDOT10]

Nevada DOT

Nevada DOT provides guidance to designers on the general application of TL3 through TL6 bridge railings in its Structures Manual.[NDOT08] TL1 and TL2 bridge rails "have no application in Nevada." Specific warrants for TL3 and higher bridge railings are not provided but the following general application guidance is offered for each test level:

- TL3 bridge railing is the minimum acceptable performance level. It may be used for roadways with "very low mixtures of heavy vehicles and with favorable site conditions."
- TL4 bridge railing is the minimum performance level for bridges on the NHS system. It may be used on high-speed highways, freeways, expressways and Interstates with a mixture of trucks and other heavy vehicles.
- TL5 bridge railing is "for a special case where large trucks make up a significant portion of the vehicular mix" and may only be used when approved by the Chief Structures Engineer.
- TL6 bridge railing is "for a special case where alignment geometry may require the use of an extra height rail" and may only be used when approved by the Chief Structures Engineer.

NDOT typically uses the 42-inch high F-shaped concrete barrier but the 32-inch high version may also be used when applicable.[NDOT08]

Washington DOT

In its Bridge Design Manual Washington DOT requires the use of at least a TL4 bridge railing on all new bridges with some exceptions. [WSDOT08] TL5 bridge railings are required under the following conditions:

- "T intersections on a structure.
- Barriers on structures with a radius of curvature less than 500 ft, greater than 10% truck traffic, and where approach speeds are 50 mph or greater.

Particular systems identified as acceptable include the F-shaped and vertical face concrete bridge railings. Washington DOT systematically improves or replaces existing deficient bridge railings within the limits of roadway resurfacing projects by "utilizing an approved crash tested rail system that is appropriate for the site" or designing a new system. Approved systems are detailed in the Bridge Design Manual and include TL2 through TL4 retrofit designs. [WSDOT08]

International Specifications

A number of European countries have developed bridge railing selection criteria and these are based on the crash testing standards and containment levels defined in European Normative 1317 (EN 1317). The basic containment levels are described herein in order to provide some basis of comparison between the US AASHTO GSBR/Report 350/MASH test levels and the EN 1317 containment levels.   EN 1317 includes four containment levels; containment "T" for low-angle containment consistent with many temporary applications; containment "N" for the normal level on most roads; containment H for high-containment levels and the H4 level for very high containment. Containment level T is not appropriate for selecting bridge railings but the other three containment levels are shown below in Table 8 with the nearest MASH test level in terms of the target energy. The MASH and EN 1317 testing requirements are different so the EN 1317 containment levels do not correspond exactly to the MASH test levels but for selection guideline comparison purposes the equivalences shown in Table 8 should be adequate. There is no MASH energy level similar to EN 1317 containment level H3 so in later tables and in this discussion H3 barriers are referred to as TL4+ (i.e., between TL4 and TL5).

Table 8. EN 1317 Containment Levels Pertaining to Bridge Railings with the nearest MASH Test Level.

Containment Level	Acceptance Tests	Containment Energy Level		Nearest MASH Test Level	Minimum MASH Energy Level	
		(kJ)	(ft-kips)		(kJ)	(ft-kips)
N1	TB31	370	273			
N2	TB32/TB11	700	516	TL2	429	316
H1	TB42/TB11	1,890	1,393	TL3	876	645
H2	TB51/TB11	2,458	1,811	TL4	3,125	2,305
H3	TB61/TB11	3,951	1,087			
H4a	TB71/TB11	4,890	1,656			
H4b	TB81/TB11	6,194	4,565	TL5	8,889	6,556

Austria

Specifications for choosing bridge railings in Austria are contained in RVS 15.04.71(15.47). The EN1317 containment levels are specified based on the highway type and certain characteristics of the roadway as summarized in Table 9. As shown in Table 9, the basic or default bridge railing containment level for freeways (i.e., divided high-speed highways) is EN1317 containment level H2 which is more or less equivalent to Report 350 TL4. For certain geometric conditions like grades on long bridges, small-radius horizontal curves, bridges with no emergency lanes and bridges that cross over high-density populated areas or other transportation facilities, the containment level is increased. The highest containment level specified is EN1317 H4b, which is roughly equivalent to Report 350/MASH TL5.

The guidelines for bridges on secondary roads (i.e., lower speed undivided roadways) have a similar pattern, although the basic containment level is EN1317 N1 which is broadly similar to Report 350 TL2. The containment level can be increased based on the geometry of the bridge and land use up to a containment level of H2 (i.e., roughly TL4).

Table 9. Austrian containment level selection guidelines.

Roadway Characteristics	**EN1317 Containment Level**
Freeways	
Normal case	H2
Upgrade > 4% for a lengths > 400m	H3
Tight horizontal curves	H3
Roads with no emergency lane	H3
Bridges over important or protected areas	H3
High-density populated areas	H3
Bridges over railroads with train speed ≥70km/h	H4
Bridges over railroads with train speed <70km/h	H2
Secondary Roads	
Normal case	N1
Upgrade > 6% for a length > 250m	N2
Tight horizontal curves	N2
Bridges over important or protected areas	H1
High-density populated areas	H1
Bridges over railroads with train speed ≥70km/h	H4
Bridges over railroads with train speed <70km/h	H2

Canada

Within Canada, each Province largely determines its own road design policy. The Alberta province Infrastructure and Transportation Roadside Design Guide , for example, references the 1989 AASHTO Guide Specification and the FHWA Heavy Vehicle Guidance discussed above. [Alberta11, AASHTO89; FHWA04] Using these documents as references, the Alberta Province guides designers as summarized in Table 10 for new and retrofit bridge railing installations.

Table 10 Alberta Canada Roadside Design Guide Bridge Rail Specifications

Test Level	**Application**
TL2	For use on local roads.
TL4	Preferred bridge rail for most applications.
	Urban bridges with cyclists.
	Urban areas where aesthetics are important.
	Short bridges (i.e., length < 20 m).
TL5	Use when high AADT with heavy truck volumes and/or high structure dictates higher test level.

Appendix C1 Section HC1.1 of the Alberta Roadside Design Guide specifically addresses upgrading existing bridge railings. [Alberta11a] Alberta employs a cost-benefit analysis procedure, outlined in Appendix C1, to determine when to upgrade existing bridge railings. Using encroachment rates and lateral extent probability, adjustments for horizontal and vertical alignment, and adjustments for bridge height and occupancy (i.e., Table 11), retrofit alternatives are considered for the remaining design life of the bridge. The null alternative (i.e., doing nothing) is also considered. The most cost-beneficial alternative is then implemented.

The Alberta procedure appears to be largely based on the 1989 AASHTO GSBR and BCAP with alterations made specific to the circumstances in Alberta. The basic equation used is:

$$PWCC = R \cdot k_c \cdot k_g \cdot P \cdot k_m \cdot k_s \cdot AC \cdot L \cdot KC/100$$

where:

- $PWCC$ = Present worth of the collision costs for one side of the bridge,
- R = Base encroachment rate in encroachments/km/yr/side,
- k_c = Highway curvature adjustment factor (i.e., unitless),
- k_g = Highway grade adjustment factor(i.e., unitless),
- P = Lateral encroachment probability,
- k_m = Multi-lane adjustment factor (i.e., unitless),
- k_s = Bridge height and occupancy factor (i.e., unitless),
- AC = Cost per collision for severity index,
- L = Length of bridge railing and
- KC = Present worth and traffic growth factor.

The base encroachment rate, horizontal curve and grade factors, which are provided in tables in the Alberta specification, are all the same as used in BCAP with the exception they are reformulated into SI units. The factor P, the lateral extent of encroachment, is similar to the approach used in BCAP and RSAP. The table is organized by shoulder width and design speed. Since bridge railings are continuous and shoulders are usually not much more than a lane-width, the lateral extent of encroachment calculation is easily converted into a simple factor. The multi-lane factor accounts for encroachments from other lanes. The bridge height and occupancy adjustment factor (i.e., k_s) is shown in Table 11. The adjustment can be as high as 2.85 for bridges over 75 feet high that span over high-occupancy land-use areas. The KC factor lumps together both the present worth factor and traffic growth factors based on an assumed four percent discount rate and two percent traffic growth and the target project life.

One of the more interesting features of the Alberta selection guidelines involves the selection of the accident cost. Appendix C2 provides a table of all the bridge railings accepted for use in Alberta and there is a table with the severity index for each bridge railing by the speed limit of the road the bridge is located on. A portion of the table is shown in Figure 36. The user of these specifications would select a particular bridge railing, find the severity index for the intended speed limit and then use that severity index to lookup the accident costs.

Once the present worth of the accident costs and the present worth of the upgrading costs are known for each alternative, the alternative with the smallest total present worth is selected.

$$TPW = PWCC + PWUC$$

where:

PWCC = Present worth of the crash costs (i.e., see equation above),
PWUC = Present worth of the up-grade costs and
TPW = Total present worth.

Table 11. Bridge Height and Occupancy Factors

Bridge Height Above Ground (m)	Low Occupancy Land use	High Occupancy Land Use
≤5	0.7	0.7
6	0.7	0.8
7	0.7	0.9
8	0.7	1
9	0.8	1.15
10	0.95	1.25
11	1.05	1.35
12	1.2	1.5
13	1.3	1.6
14	1.45	1.7
15	1.55	1.85
16	1.7	1.95
17	1.8	2.05
18	1.95	2.2
19	2.05	2.3
20	2.2	2.4
≥24	2.7	2.85

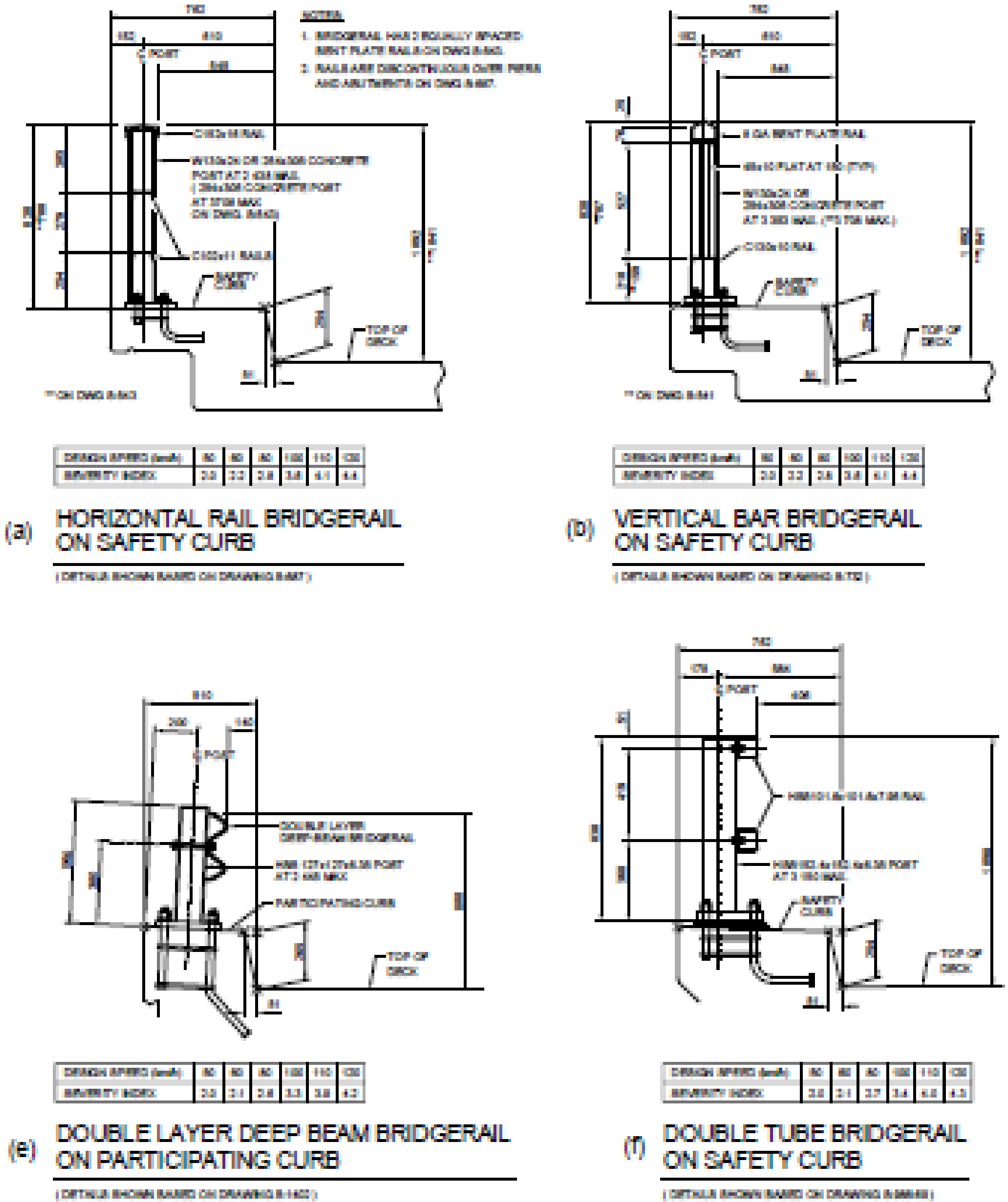


Figure 36. Portion of the Alberta bridge rail severity index selection table. [Alberta11b]

Germany

The bridge railing selection guidelines for Germany are provided in RPS 2008 in section 3.5.1.1 and are summarized below in Table 12. The German selection guidelines generally segregate bridges into those that pass over sensitive areas, populated areas or other transportation infrastructure. When the bridge crosses over a more sensitive area, the containment level is increased. The basic containment level for low speed roads is H1 which is essentially like Report 350 TL3. The highest containment level specified is H4b which is well in excess of Report 350 TL5 or, for that matter, TL6. The selection is based on the area crossed over and the speed and traffic volume of the roadway.

Table 12. German bridge railing selection guidelines.

Bridge Characteristics	**Traffic Characteristics**			
	Speed > 100 km/h	**Speed ≤ 100 km/h AADT > 500 vpd**	**Speed ≤ 100 km/h AADT ≤ 500 vpd**	**Speed ≤ 50 km/h**
Bridges with dangerous areas beneath like: • Explosive chemical plants , • High-density areas, • High-speed rail tracks with speeds > 160 km/h • Two-lane roads	H4b	H2	H2	H1
Other cases	H2	H2	H1	

Italy

The Italian Ministry of Infrastructures and Transportation instructs designers on the selection of roadside barriers and bridge railings in a decree titled "Update of technical instructions for the design, approval and use of road safety barriers, and technical regulations for testing road safety barriers."[Italy11] The decree references the containment levels specified in EN 1317, traffic is subdivided into four categories by volume and heavy vehicle percentage, as shown in Table 13. Table 14 is then used to determine the appropriate application of bridge railing.

Table 13. Composition of Traffic by Category. [Italy11]

Type of traffic	ADT*	% of vehicles weighing > 3.5t
I	1000 or fewer	Any
I	>1000	5 or lower
II	>1000	5 to 15
III	>1000	>15

*ADT: Average Daily Traffic annually in both directions.

Table 14. Bridge Railings and Other Roadside Barriers by Traffic Category. [Italy11]

Type of barrier	Type of traffic	Medians	Roadside Barriers	Bridge Railings (1)
Freeways (A) and main state and local highways (B)	I	H2	H1	H2
	II	H3	H2	H3
	III	H3-H4 (2)	H2-H3 (2)	H3-H4 (2)
Extra-urban and secondary highways (C) and ring roads (D)	I	H1	N2	H2
	II	H2	H1	H2
	III	H2	H2	H3
Urban city street (E) and local roads (F)	I	N2	N1	H2
	II	H1	N2	H2
	III	H2	H1	H2

(1) Bridges or viaducts are defined as structures crossing a space of more than 10 meters; structures crossing less space are considered equivalent to roadsides.
(2) The choice between the two classes is decided by the project engineer.

A review of Table 14 indicates that freeways generally have a higher performance level bridge railing for any given traffic category then the other roadway types. Generally speaking, EN1317 H2 barriers are more or less equivalent to TL4. The H4 barriers are approximately equivalent to TL5 barriers in the U.S. Therefore, the basic default condition in Italy is a TL4 bridge railing with TL5 bridge railings used on roadways with AADTs greater than 1000 vehicles/day and a percent of trucks greater than 15.

United Kingdom

Unfortunately, the UK has experienced a number of dramatic and catastrophic crashes involving vehicles leaving bridges and falling onto railway tracks. The 2001 Selby rail crash, for example, occurred on 28 February 2001 in Great Heck near Selby in North Yorkshire. A Land Rover towing a trailer struck an approach guardrail and came to rest below the bridge on the East Coast Main Line. The vehicle was struck by the Newcastle to London King's Cross train at over 120 mi/hr resulting in the derailment of the train; 10 people were killed and 82 were injured in the crash. [Wainwright02]

While the Selby crash involved the approach guardrails to the bridge, there have been other very similar cases involving penetrating bridge rails over rail lines in the UK. For example, a concrete mixer truck penetrated a brick bridge parapet near the town of Oxshott in Surey, England at about 3:30 pm on November 5th, 2010. [RAIB11] The truck landed on top of the sixth car of the Guildford to London Waterloo train. The truck driver was seriously injured; one train passenger was seriously injured and five other sustained minor injuries in the crash. Fortunately, no one was killed in the crash.

As a result of the Selby and Oxshott crashes, the UK Ministry of Transport re-evaluated its design guidelines for barriers. The intent was to develop more explicit detailed risk-based design guidelines. The United Kingdom has implemented a performance based design process, supported by software coded in Excel and based on the Design Manual for Roads and Bridges. [UK06] This process applies to the selection of bridge railings with some pre-established minimum containment levels which reference EN1317. These minimum Containment Levels are provided for bridge railings over or adjacent to roads unless the Road Restraint Risk Assessment Process (RRRAP) (i.e., the UK cost/benefit analysis process similar in some senses to RSAP) dictates a higher containment level:

- On roads with a speed limit of 50 mph or more:
    1. Normal Containment Level = N2
    2. Higher Containment Level = H2
    3. Very High Containment Level = H4a
- On roads with a speed limit of less than 50 mph:
    1. Normal Containment Level = N1
    2. Normal Containment Level = N2
    3. Higher Containment Level = H2
    4. Very High Containment Level = H4a

Other than in Northern Ireland, at minimum a very high containment Level (H4a) bridge railing is used on new bridges and structures carrying a road over or adjacent to a railway. The "highest practicable containment level that can be achieved without undue cost," as determined by the UK RRRAP, is provided for retrofit bridge railings over or adjacent to a railway. Within Northern Ireland, the minimum bridge railing containment level is normal containment level (N2) when the road is over or adjacent to a railway. When a higher containment level is justified through the use of the performance-based design process (i.e., RRRAP), "the level of provision must be confirmed with the

Overseeing Organization and the Railway Authority." On an existing bridge that is not over or adjacent to a railway, the Containment Level requirements are as follows:

- Where the existing bridge or structure can support a bridge railing with a containment level derived from the RRRAP, this level of containment must be provided.
- Where the existing bridge or structure cannot meet the containment level derived from RRRAP, further assessment is conducted to determine the level of containment possible without strengthening.
- If the risks associated with the provision of the lower level of containment determined through the RRRAP, then the lower containment level is provided. If not, the bridge is improved to provide a higher containment bridge railing.

Summary

Table 15 through Table 18 shows a summary of all the bridge selection guidelines discussed in the previous sections. These tables summarize the guidelines by highway function, heavy vehicle accommodation, a combination of highway design selections and by geometric design factors.   The European standards have been included in these summary tables using the following approximate equivalency between EN 1317 containment levels and Report 350 test levels.

- EN1317 containment level N1 is approximately equivalent to TL3;
- EN1317 containment level H2 is approximately equivalent to TL4; and
- EN1317 containment level H4a is approximately equivalent to TL5.

Most States allow TL2 railings for non-NHS roadways while TL4 is generally the preferred minimum for NHS roadways. TL5 railings are generally specified for roads with heavy truck traffic, however the different states define heavy truck traffic differently and a number of States recommend TL5 bridge railings on all Interstate highway applications. TL5 railing are also recommended for some sharp horizontal curves with varying definitions of sharp, sometimes with no definition of sharp at all.

Table 15. Summary of R350 Bridge Railing Selection by Highway Function.

State/Country	Based on present worth of crash and upgrading costs (min).	Min used for retrofit designs.	Min that existing may remain.	Min for Non-NHS roadways	Min for NHS roadways	Min for permanent installations.	Used for all new bridges	Min for all Secondary highways	Local and collector roads with favorable site conditions.	High-speed highways, expressways and interstates	For exit ramps.	Work zones.
FL DOT						TL4						TL2
NY DOT									TL2	TL4		TL2
NJ DOT				TL1	TL4						TL5	
RI DOT									TL4	TL5		<TL4
MA DOT				TL2	TL4					TL5	TL5	
OH DOT			TL2			TL3						
NC DOT				TL2	TL3							
IL DOT					TL4							
NV DOT					TL4	TL3	TL4					
WA DOT		TL2										
Austria						TL4			TL2			
Alberta	TL3											
Italy								TL4	TL4			
UK	TL2											

Table 16. Summary of R350 Bridge Rail Selection Guidelines for Heavy Vehicle Accommodation.

State/Country	Small number of heavy vehicles is expected.	The volume of truck traffic is unusually high.	Heavy vehicle containment	High-speed highways w/ high traffic volume and %T	Greater than 10% T	A vehicle penetrating would cause high risk.	Bridge crossing over protected areas	Freeways with bridges crossing over rail lines with train speed ≥ 50 mi/hr
FL DOT		TL5				TL6		
NY DOT	TL2			TL5				
NJ DOT			TL5					
RI DOT	<TL4	TL5		TL5				
IL DOT			TL5					
NV DOT	TL3	TL5						
WA DOT					TL5			
Austria							TL4+	TL5

Table 17. Summary of R350 Bridge Rail Selection Guidelines by Combination of Selectors.

State/Country	High Protection Area where Dc > 5° and Speed > 40 mph.	Speed limit ≤ 30 mi/hr and dangerous area under bridge	30 mi/hr ≤ Speed limit ≤ 60 mi/hr, ADT ≤ 500 vpd and no special situation under bridge	30 mi/hr ≤ Speed limit ≤ 60 mi/hr, any ADT, and a dangerous area underneath the bridge	30 mi/hr ≤ Speed limit ≤ 60 mi/hr, ADT > 500 vpd and No special situation under bridge	60 mi/hr ≤ Speed limit and a dangerous area under bridge	ADT <1000 vpd or Trucks ≤ 5 %	ADT > 1000 vpd and Truck > 5 %
MN DOT	TL5							
RI DOT	TL5							
Germany		TL3	TL3	TL4	TL4	TL5		
Italy							TL4	TL4+

Table 18. Summary of R350 Bridge Rail Selection Guidelines by Geometric Design Considerations.

State/Country	Alignment has sharp horizontal curve.	For horizontal curves of less than 1000 feet.	Radius of curvature less than 500 ft	Where alignment may require extra height.	Upgrades > 4% lasting more than 400 m	Roads with no emergency lanes	Traffic Only (i.e., no bikes or peds).	Bridges with designated bike path	'T' intersections on bridge.	For speeds $\leq$ 40 mph.	Speeds $\geq$ 50 mph
FL DOT	TL5										
MN DOT							TL4	TL5		TL2	
NJ DOT		TL5									
RI DOT	TL5										
NC DOT										TL2	
IL DOT		TL5									
NV DOT				TL6							
WA DOT			TL5						TL5		TL5
Austria	TL4+				TL4+	TL4+					

Crash Data Studies

FHWA Narrow Bridge Study

The FHWA sponsored research at Southwest Research Institute in the early 1980's to examine crash characteristics at narrow bridge sites. [FHWA83] The research, which was performed by Mak and Calcote, was intended to determine the extent of the crash problem associated with narrow bridge sites and collect statistics on the frequency, severity and site characteristics. Crashes on 11,880 bridges were collected from five states and a subfile of 1,989 bridge cases were identified for more detailed analysis. Another subfile of 124 bridge crashes were selected for in-depth analysis. The data included not only bridge railings but also approach guardrail, transitions and approach guardrail terminals so it is sometimes difficult to isolate just the bridge railing effects.

Since the study was performed in the early 1980's based on data that had been collected primarily in the 1970's, most of the structures were built prior to the 1965 AASHTO Bridge Design Specifications. The average construction date in the population file was 1954. The results of this study, therefore, do not represent more modern crash tested bridge railings. There was relatively little in the narrow bridge data to indicate the proportion of rollovers and penetrations and, in any case, that data would not be reflective of the types of bridge railings that are commonly installed today.

Table 19 shows the distribution of barrier performance in terms of the number and percent of vehicles that were redirected, over-rode, vaulted or penetrated the barrier. Unfortunately, the authors did not separate out different barrier types so it is believed that the data in Table 19 include bridge railings, transitions and guardrails. Similarly, it is not clear if there is a distinction between rollover back onto the roadway or rolling over the bridge railing and off the bridge. In any case, nearly 75 percent of vehicle collisions resulted in redirection and the remaining 25 percent were a combination of penetrations, vaults and rollovers. If it is assumed that Table 19 represents mostly bridge railing impacts, this would suggest that pre-1965 bridge railings resulted in about four percent penetrations and 18 percent rollovers and vaults.

Table 19. Barrier Performance in Narrow Bridge Crashes. [FHWA83]

Barrier Performance	1st Impact		2nd Impact		3rd Impact		Total	
	No.	%	No.	%	No.	%	No.	%
Redirected	87	73.1	53	79.1	16	76.2	156	75.4
Overrode	12	10.1	9	13.4	5	23.8	26	12.6
Vaulted	10	8.4	1	1.5	0	0.0	11	5.3
Penetrated	5	4.2	3	4.5	0	0.0	8	3.9
Other	5	4.2	1	1.5	0	0.0	6	2.9
Unknown	5	--	1	--	1	--	7	--
Total	124	100.0	68	100.0	22	100.0	214	100.0

Interestingly, 77 percent of the crashes involved multiple impacts where the vehicle struck and re-struck the bridge railing. While the percent redirected stays more or less around 75 percent, the percent of over-rides increases from 10 to 23 percent and the percentage of penetrations decreases from 4 percent to zero. This is probably reasonable since there would be less energy available for creating a structural failure in each subsequent crash (i.e., less chance of penetration) but the impact angles and yaw rates probably increase for subsequent impacts which might promote overrides.

Mak and Calcote found that the crash severity increased as the bridge length, percent of shoulder reduction and speed limit increased. The departure angle (i.e., encroachment angle) was 15 degrees or less for more than 61 percent of the cases and, as would be expected due to the small distance between the edge of travel and the bridge railing, the distance from departure to impact was less than 50 feet in 78 percent of the cases. Mak and Calcote provided a great deal of information about the encroachment conditions at narrow bridge sites including the encroachment speed, angle, lateral extent and other impact conditions.

NCHRP 22-08

NCHRP Project 22-08 was the most comprehensive analysis of bridge rail performance available to-date in the literature. [Mak94] The purpose of NCHRP 22-08 was to evaluate the appropriateness of AASHTO's 1989 GSBR. Project 22-08 attempted to determine the safety performance of existing bridge rails by examining more than 4,500 bridge rail crashes across the state of Texas. Hardcopies of the accident reports were examined for all reported bridge departures (i.e., penetrations and rollovers). Stratified random samples of accident reports from all other crashes were used as a quality control check to identify the frequency of coding errors. Likewise, the age of bridge rails were examined for all bridge departures and random samples were used to identify characteristics of railings that retained impacting vehicles. This study found remarkably high bridge rail crash severities for impacts involving vehicles retained on the bridge. In fact, a total of 365 (8.1%) serious injury and fatal (A+K) crashes were associated with vehicles retained on the bridge compared to 78 (A+K) (1.7%) crashes arising from a vehicle penetrating through or going over a bridge rail. In other words, more than 4.5 times more serious injury and fatal crashes occurred when a vehicle was contained on a bridge then when the vehicle penetrated through or vaulted over the railing. This ratio of serious injury and fatal crashes when the vehicle is retained versus departing from the bridge was virtually unchanged when the analysis was limited to interstate freeways which would presumably have more modern bridge rails. These findings may indicate that societal costs of bridge rail accidents are more strongly related to bridge rail performance during redirection crashes than to the number of vehicles leaving the bridge. On the other hand, this also may be more a reflection of the generally rural character of many Texas roads.

This may actually make sense since if the consequences of penetrating the railing are limited to the truck and its occupants, there is much more potential for harm when the vehicle actually stays on the road where it will interact with other vehicles. This demonstrates that the potential for harm from a re-direction or a penetration/vault is very sensitive to the land use around the bridge structure. If the bridge does not pass over a transportation facility or urbanized area, the consequences of leaving the bridge for the general public would be less serious than remaining on the bridge.

The importance of serious injury and fatal crashes associated with a vehicle being retained on the bridge was further reinforced when the age of the bridge rail was taken into consideration. Bridge railings designed to more modern standards, AASHTO's 1965 or later Bridge Specifications, were found to have bridge departure rates (i.e., both rolling over the bridge railing and penetration of it) of approximately 2.9% compared to 5.9% for all vehicle types in the database as shown in Table 20. The results for trucks were even more dramatic as shown in Table 20; single-unit truck rollovers and penetrations decreased from 5.4 percent to 2.3 percent; a 57 percent reduction. Tractor-trailer truck rollovers and penetrations experienced a dramatic decrease from 24.5 percent for bridge railings designed before 1965 to 7.8 percent for those designed after 1965. Clearly, the requirements of the 1965 Bridge Specifications had a dramatic effect on reducing rollovers and penetrations of bridge railings.

Table 20. Penetration and rollover percentage in Texas bridge railing crashes. [Mak94]

Bridge Railing Design Year	All Vehicle Types (%)	Single Unit Trucks (%)	Tractor-Trailer Trucks (%)
Before 1965	5.9	5.4	24.5
After 1965	2.9	2.3	7.8
Reduction	51	57	68

The bridge departure rate was further reduced when hardcopies of accident reports were carefully reviewed. This hard-copy analysis found that only a third of the reported bridge departures actually involved a vehicle striking a bridge rail. The remaining crashes were found to involve vehicles penetrating through or going over a bridge approach transition, a guardrail or guardrail end. Similarly, when a subset of the data were visually inspected, many of the cases coded as single-unit trucks were, in fact, pickup trucks, utility vehicles and vans. In fact, of the 53 cases where hard-copy were reviewed only 15 actually involved trucks going through or over bridge railings. When improper coding and age of the bridge rail were taken into consideration, it was found that modern bridge rails contained approximately 99 percent of the trucks striking the bridge railing; or conversely, the rollover and penetration percentage for trucks was around one percent.

Kansas Bridge Rail Study

A more recent study of bridge rail crashes in Kansas found much lower severities than reported in NCHRP 22-08. [Sicking09] This study examined all bridge rail crashes on controlled access freeways in the state of Kansas for the years 2002 through 2006. A total of 705 bridge rail crashes were identified. The combined A+K rate for bridge rail crashes was found to be 3.43% compared to 8.1% found in the Texas study. The lower crash severities observed in Kansas are believed to be related to this state's lower accident reporting threshold compared to Texas. When hardcopies of accident reports from all serious injury and fatal crashes were examined, it was found that only one of the 24 reports improperly coded a guardrail crash as bridge rail impact. Of the 23 remaining serious injury or fatal crashes involving a bridge rail, only one involved a vehicle going through or over the railing. In fact, this crash involved a tractor trailer breaking through a bridge rail and surprisingly, the driver was not killed when his truck fell 50 feet to land on railroad tracks below the overpass. The three reported fatal bridge rail crashes included a passenger car rollover, a light-truck that lost its driver side door, and a passenger's head extending out of the window to strike a concrete barrier. Based on the more than 20:1 ratio between serious injury and fatal crashes involving containment versus penetration, it appears that vehicles leaving a bridge are not a major source of bridge rail crash costs in Kansas. Kansas' controlled access freeways primarily utilize open concrete, New Jersey, and F shape concrete bridge rails. With the exception of bridges over railroad tracks, almost all existing Kansas bridge rails are 32 inches high and fall into the TL3 category under the new MASH criteria.

The relatively low capacity associated with most Kansas bridge railings (i.e., Kansas generally uses MASH TL3 bridge railings) and the infrequency of serious injury and fatal crashes associated with vehicles departing these bridges makes it very clear that bridge rail selection guidelines should not be based solely upon barrier capacity.

Analysis Methods for Bridge Railing Selection

There is a surprisingly long history of using benefit-cost encroachment-based computer programs in roadside safety. The 1977 Barrier Guide presented a hand-calculation method based on work by Glennon but it was not particularly practical for roadside design practitioners since there was a lot of tedious hand calculation required. In 1989, AASHTO revised, updated and expanded the 1977 Barrier Guide transforming it into the Roadside Design Guide. [AASHTO89] Appendix A of the Roadside Design Guide included a revision of the cost-effectiveness procedures and provided a computer program called Roadside to ease the calculation burden on designers and policy makers. BCAP was largely based on the Roadside method with a number of enhancement and improvements intended for use in selecting bridge railings using a cost-benefit encroachment estimation procedure. [AASHTO89]

In their day, Roadside and BCAP were innovative implementations of risk-based probabilistic roadside cost-benefit design. Of course, as computer applications became more sophisticated and additional research was performed to refine and improve encroachment models, severity indices and other aspects of the procedures, it became apparent that a new computer program was needed. The resulting program, the Roadside Safety Analysis Program (RSAP), was completed in 2003 and documented in NCHRP Report 492 by Mak and Sicking. [Mak03] Additional research on measured vehicle trajectories during encroachments and the replacement of severity indices with the equivalent fatal crash cost ratio (EFCCR) as well as continued advancements in computers culminated in the most recent update to RSAP in 2012, RSAPv3. [Ray12] This current research is based on simulations performed using RSAPv3, however, the predecessors to RSAPv3are discussed here because each of these software tools represent individual steps forward in the development of bridge rail selection guidelines.

BCAP

Like Roadside, BCAP assumes an encroachment rate of 0.0005 encroachments/mi/yr per edge of roadway and could be modified to account for grade and curvature of the roadway with adjustment factors. The encroachment model investigated encroachments by 13 vehicle types leaving the roadway at 10 different speeds and up to 12 different angles. The total crash cost associated with a design alternative was calculated by summing the crash costs for each encroachment condition multiplied by their associated probabilities of occurrence. The crash costs for each encroachment were estimated based on the severity of the encroachment (i.e., the consequences of impacting the bridge rail at the prescribed encroachment conditions).

BCAP used a severity index (SI) scale of zero to ten to define the severity of a predicted barrier collision. Each SI had an assumed distribution of accident outcomes ranging from property damage only (PDO) to fatality. For redirection impacts, the SI was assumed to be linearly related to the lateral acceleration of the vehicle. BCAP also assumed that any bridge railing with acceptable crash test performance would have the same severity index for redirectional collisions. For a barrier penetration, the SI was assigned a value of 7.0. For a rollover on the traffic side of the barrier, the severity was linearly correlated to impact speed.

The probability density function (PDF) for encroachment speed ranged from zero to 15 mph above a reference speed which was taken as 0.9 times the highway design speed. For a given encroachment speed, the encroachment angles were varied in three degree increments from zero up to a maximum of 36 degrees. The PDF was assumed to vary linearly between these two points. A model was used to determine the maximum angle a vehicle can leave the roadway without skidding or overturning. The model included consideration of barrier offset distance, encroachment speed, tire-pavement friction, vehicle stability, and minimum turning radius. In cases in which the model precludes higher angle encroachments, the encroachment angle PDF was truncated and adjusted.

BCAP assumed a straight line encroachment trajectory. The maximum extent of lateral encroachment was estimated using a constant deceleration rate of 13 ft/sec^2, which is equivalent to a braking friction of 0.4. Since BCAP was intended only for bridge railing applications, there is no representation of the roadside (i.e., no slopes or other off-road hazards) and the lateral extent of encroachment is relatively unimportant since bridge rails are by definition placed relatively close to the edge of the roadway.

As discussed earlier, BCAP estimates the force imposed on the bridge railing by each collision using the speed, angle and mass of the encroaching vehicle. This force estimate is then compared to the assumed capacity of the bridge railing. If the capacity is less than the impact force, the bridge rail is assumed to be completely penetrated and the vehicle is assumed to fall off the bridge. If the impact force is less than the capacity, redirection is assumed and the vehicle conditions are checked to see if rollover is likely. As discussed earlier with respect to the 1989 AASHTO GSBR recommendations, BCAP was found by Mak and Sicking to over predict barrier penetrations and under predict rollovers by a considerable margin.

The penetration model used in BCAP was based on work by Olsen in NCHRP Report 149. [Olsen74] Olsen suggested that the lateral force imparted by the vehicle to the barrier could be approximated as:

$$F_{lat} = \frac{W\,V^2\,sin^2\theta}{2g(A\ \sin\theta - \frac{B}{2}(1-\cos\theta) + D)}$$

where:

F_{lat}	= The average lateral deceleration of the vehicle,
W	= The weight of the vehicle in lbs,
V	= The vehicle impact velocity in ft/sec,
Θ	= The impact angle,
A	= The distance from the front of the vehicle to the center of mass in ft,
B	= Vehicle width in feet and
D	= Lateral deflection of the barrier in feet.

BCAP generates a set of encroachment conditions (i.e., speed, angle and vehicle type) and this lateral force can then be calculated based on those assumed impact conditions. If the lateral impact force is greater than the capacity of a barrier, the barrier is assumed to have failed structurally. While Olsen's model is a good simple estimator it certainly has its limits. First, it is based on estimating the impact force when damage is more properly related to strain energy. Unfortunately, while impact energy is easy to calculate (i.e., ½ mv^2), the strain energy capacity of a barrier is quite difficult to calculate at least in some simplified form. Also, in developing the 1989 GSBR recommendations, it was assumed that the barrier deflection would always be zero. This is probably reasonable for rigid concrete barriers but it has the effect of under estimating the capacity of post-and-beam types of bridge railings. Another flaw with this penetration model, at least with respect to its use in BCAP, is that once capacity has been reached it is assumed the barrier is totally compromised when in fact the capacity load is really just the beginning of the failure process. The barrier may often contain and redirect the vehicle even though there are structural failures; in other words, reaching capacity does not necessarily mean the vehicle will penetrate the barrier.

The rollover algorithm only is activated if the bridge railing is not penetrated. BCAP first checks to see if the capacity has been reached. If capacity has been exceeded, the vehicle penetrates the railing. If capacity has not been exceeded, the vehicle is assumed to be redirected and the rollover algorithm is activated. The rollover condition in the original BCAP is:

$$V_{cr} = \frac{\sqrt{\frac{g}{2}\left[\frac{5B^2}{4}+\frac{H_{cg}^2}{144}+\frac{(H_{cg}-H_b)^2}{36}\right]\left[\sqrt{\frac{B^2}{4}+\frac{(H_{cg}-H_b)^2}{144}}-\frac{(H_{cg}-H_b)}{12}\right]}}{\frac{(H_{cg}-H_b)\sin\theta}{12}}$$

where:

V_{cr}= The velocity in ft/sec that the vehicle would rollover,
g = The acceleration due to gravity (i.e., 32.2 ft/s^2),
H_{cg} = The height of the vehicle center of gravity in ft,
H_b = The height of the barrier in feet and
Θ = The impact angle.

This formulation assumes that the vehicle forces act at the center of gravity of the vehicle and that the barrier forces act at the very top of the barrier. Mak and Sicking found that this equation yields critical velocity estimates that are too high so BCAP seldom predicted a rollover.

Mak and Sicking modified the model by assuming the barrier forces act at the vehicle axle rather than top of the barrier and that the truck would rotate about the top of the barrier when the truck deck settled onto it during the rollover. The improved impulse-momentum model is given by:

$$V_{cr} = \frac{\sqrt{\frac{2g}{12}[(d+H_b-H_{cg})]}}{R\sin\theta}\left[\frac{H_{cg}-H_b}{12}+\frac{12R^2}{(H_{cg}-H_f)}\right]$$

where the terms are as before in addition to:

d = Distance from the vehicle c.g. to the bottom edge of the truck frame in inches,
H_f = Height of the center of the truck axle in inches,
R= The radius of gyration of the truck and its load about the bottom corner of the truck frame.

This model was validated to some extent with HVOSM and NARD and resulted in lower critical velocities and more rollovers in the BCAP analyses. As discussed earlier, the improved rollover algorithm and adjustments to the barrier capacity performed by Mak and Sicking improved the estimates of BCAP but BCAP still predicted many more crashes than comparison to the real-world data available at the time indicated.

Both of these rollover models completely ignore the effect of barrier shape on vaulting and rollover by vehicles with c.g. heights lower than the barrier height. For example, many passenger cars vault over safety shaped barriers even though the height of the passenger car c.g. is lower than the barrier height. The reason is that the shape of the barrier in some shallow impact angles has the effect of launching the vehicle over the barrier. This is not accounted for in either model.

BCAP used the crash costs shown in Table 21 which, by today's standards, are very low. There is no explicit provision in the 1989 AASHTO GSBR for updating these costs or adjusting them for inflation.

Table 21. Crash Costs used in BCAP. [AASHTO89]

Crash Severity	Average Cost
Fatal	$500,000
Severe	$110,000
Moderate	$10,000
Slight	$3000
Property damage only (level 2)	$2500
Property damage only (level 1)	$500

In summary, the BCAP was innovative and ground-breaking in many ways in its time. It used a benefit-cost approach to develop the guidelines and presented a systematic method for selecting the appropriate bridge railing. Unfortunately, some of the data in BCAP was flawed and some of the algorithms were overly conservative. The general approach was a reasonable way to select bridge railings but the assumptions, lack of data and lack of validation resulted in unrealistic recommendations.

RSAP

The Roadside Safety Analysis Program (RSAP) is a computer program for performing encroachment-based cost-benefit analyses on roadside designs. A key step in performing such analyses is to estimate the frequency and severity of roadside crashes for a particular roadside design where the design encompasses highway geometric features like the horizontal curvature and grade as well as the roadside features like the location and type of guardrails, the shoulder widths and the slope of the roadside. Once the frequency and severity of crashes has been estimated, the cost can be found by mapping the frequency and severity into units of dollars given the average societal cost of each expected crash. A roadside design that results in a smaller societal cost is, therefore, a safer and better design. If the reduction in crash costs over the design life of the improvement are greater than the construction and maintenance costs of the improvement the design is cost-beneficial and should be constructed. On the other hand, if the reductions in crash costs are less than the construction/maintenance cost of the improvement the project probably is not worth pursuing.

Estimating the frequency and severity of crashes for a given roadside design can be challenging since all the variables are probabilistic in nature and many are not well known or understood. For example, vehicles will leave the roadway (i.e., encroach) at a variety of speeds, angles and orientations; vehicles will leave the road at various points along the road segment and the path taken by the vehicle off the road will depend on driver steering and braking input, . Likewise, not all vehicles that leave the road will strike an object so there is a probability distribution associated with the likelihood of striking an object once the vehicle leaves the road. Even when a vehicle does strike an object like a guardrail, sign support or tree on the roadside, the severity of the crash can vary from no-injury to one with multiple fatalities. Since estimating the frequency and severity of roadside crashes involves several conditional probabilities, methods like RSAP are really risk-based probabilistic analysis tools where mathematical models of probabilities and risk are manipulated in order to estimate the frequency and severity of crashes.

RSAP uses these four modules to assess the cost-effectiveness of a design:

- Encroachment Module,
- Crash Prediction Module,
- Severity Prediction Module, and
- Benefit/Cost Analysis Module.

The encroachment probability model is built on a series of conditional probabilities. First, given an encroachment, the crash prediction module then assesses if the encroachment would result in a crash, $P(C|E)$. If a crash is predicted, the severity prediction module estimates the severity of the crash, $P(I|C)$. The severity estimate of each crash is calculated using crash cost figures so the output is in units of dollars.

The original version of RSAP estimated the crash costs using a Monte Carlo simulation technique that simulates tens of thousands of encroachments and predicts the frequency and severity of each simulated encroachment. For each alternative, an average annual crash cost was calculated by summing the crash costs for all the simulated crashes on each segment. These crash costs were then normalized to an annual basis. Any direct costs, (i.e., initial installation and maintenance) were also normalized using the project life and the discount rate. Similar to its predecessor, the original version of RSAP used straight line vehicle trajectories and a stored table of possible angles and speeds.

The third version of RSAP (RSAPv3) was used in this research. [Ray12] Using a series of conditional probabilities, RSAPv3 first predicts the number of encroachments expected on a segment. Given an encroachment has occurred, the likelihood of a crash is assessed by examining the location of roadside features and comparing those locations to a wide variety of possible field collected vehicle paths across the roadside. If a crash is predicted (i.e., one of the possible trajectories intersects with the location of a roadside hazard), the severity is estimated and converted to units of dollars.

RSAPv3 proceeds by simulating tens of thousands of encroachment trajectories and examining which trajectories strike objects, the probability of penetration or rolling over the object and the likely severity of those collisions. The passenger vehicle trajectories used in RSAPv3 were gathered from reconstructed run-off-road crashes under NCHRP 17-22. [Mak10]

After the total crash costs and the direct costs are calculated for each alternative, the concept of incremental benefit/cost (*B*/*C*) is used to determine the cost-effectiveness of the design. The *B*/*C* ratio used is as follows:

$$\text{B/C Ratio}_{2\text{-}1} = \frac{CC_1 - CC_2}{DC_2 - DC_1}$$

Where:
B/C Ratio 2-1= Incremental B/C ratio of alternative 2 to Alternative 1
CC_1, CC_2, = Annualized crash cost for Alternatives 1 and 2
DC_1, DC_2, = Annualized direct cost for Alternatives 1 and 2

RRRAP

The Road Restraint Risk Assessment Process (RRRAP) is a software program developed in the UK to aid in the implementation of the "Design Manual for Roads and Bridges," TD 19/06. [UK06] RRRAP is a risk-assessment software tool to allow engineers to explicitly assess the risks associated with crashes and compare and evaluate different alternatives. Alternatives are compared using a benefit-cost procedure much like has been used in RSAP, BCAP and Roadside. RRRAP was coded in MS Excel using extensive macros to perform the bulk of the numerical calculations.

Broadly speaking, RRRAP is limited to "trunk" roadways (i.e., roadways that are not local roads and streets) with posted speed limits of 50 mi/hr and above and AADTs of 5,000 vehicles/day and greater. By default, RRRAP assesses the "null" conditions (i.e., no roadside safety features) with the basic EN 1317 containment level of N2. The user can then elect to explore other containment levels to determine if they are or are not cost-beneficial. Three vehicle classes are considered – "light" vehicles are all vehicles less than 3,000 lbs, "medium vehicles" are those weighing between 3,000 and 7,000 lbs and large goods vehicles are all vehicles weighing over 7,000 lbs.

RRRAP treats bridge railings a little differently than most other roadside hazards in that the only risk that is considered is the risk of breeching (i.e., penetrating or rolling over) the bridge railing. In other words, crashes where the vehicle is contained on the bridge are not considered in the risk assessment. The assumption is that the major risk for a bridge railing is the risk to third parties (i.e., either non-road users or users of other transportation facilities beneath the bridge).

The basic procedure for bridge railings in RRRAP is to start with the basic EN 1317 N2 containment level and determine the likely number of breeches for each class of vehicles. The bridge parapet breech rate is then computed by dividing the number of breeches in each vehicle category by the length of the bridge and then summing those rates to arrive at the total number breachings per year per foot of bridge. Some modification factors are applied to account for the shoulder width and type. Next the average cost of a crash is multiplied by the number of breachings considering first only the road users and then third parties.

If the risk (i.e., total penetrations/mi/yr) is less than a predetermined threshold, then the alternative is acceptable. If the risk is above the minimum threshold, the process is repeated for the next containment level and the alternatives are compared by calculating the benefit cost ratios between the alternatives until a cost-beneficial solution is achieved.

Risk Analysis

Another approach not often used explicitly in roadside safety but common in many other types of engineering fields is risk analysis. In risk analysis the risk of experiencing a particular type of event is quantified using probabilistic models. An acceptable level of risk is established over the project life and then the system is engineered to ensure that the risk in-service is below the targeted acceptable risk. For example, a transportation agency might decide that if the risk of a severe or fatal injury over the 30-year life of the project is less than 0.05 it is acceptable.

The benefit-cost method used in roadside safety is actually a risk assessment method to estimate the reduction in anticipated crash costs (i.e., the benefits) then a standard benefit-cost analysis that includes the calculated crash costs and agency costs such as construction, maintenance and repair over the life of the project. Roadside safety analysis programs like Roadside, BCAP and RSAP have always calculated the average expected cost of crashes by simulating tens of thousands of possible encroachments and then multiplying by the expected number of encroachments each year. The average crash cost is calculated as follows:

$$\overline{CC} = \frac{1}{NM}\left[\sum_{j=1}^{M} w_j \left[\sum_{i=1}^{N} CC_{ij}\right]\right]$$

where:

$\overline{CC}$ = The average annual crash cost,
CC_{ij} = The crash cost of encroachment i with vehicle type j,
W_j = The proportion of the traffic volume accounted for by vehicle type j,
N = The total number of encroachments simulated and
M = The total number of different vehicle types in the traffic mix.

Earlier roadside safety benefit-cost programs simply calculate the average annual crash cost "on the fly" without saving the crash cost of each encroachment but RSAPv3 saves individual terms of the summation so that the distribution of crash costs over the life of the project can be examined. Since the probability distribution of crash costs is saved, the risk of exceeding any particular crash cost (i.e., severity level) can be easily calculated.

Conclusions

While anecdotal from a statistical point of view, the crashes discussed earlier illustrate several interesting points. Many of the crashes involved horizontal curvature, curved on/off ramps, bridges and overpasses over other highways, hazardous materials routes, heavy vehicles, or highways with large numbers of trucks and buses. In some cases these crashes have occurred on highways specifically designated as "truck routes" or "hazardous material routes" which would seem to suggest that these highways should use barriers capable of restraining such vehicles. On the other hand, some of the crashes reported by the media or investigated by NTSB occurred at sites that would likely not have been considered to be particularly susceptible to a heavy vehicle crash.

While some of the crashes certainly occurred at sites with the three risk factors noted in the RDG and the 1989 AASHTO GSBR (i.e., adverse geometry, percent of trucks and adverse consequences of penetration), others do not. For example, the bridge railing involved in the Sherman, Texas crash was probably a non-crash tested bridge railing that had been constructed long before FHWA made crash testing of bridge railings mandatory. The Sherman, Texas crash site did not have adverse geometric characteristics, did not pass over a sensitive facility or area and was not on a highway with a particularly large percentage of trucks so it is unlikely it would have warranted a higher performance railing than most other highway applications. What was really required at that particular site was a way to identify substandard and un-crash tested bridge railings and replace them. On the other hand, the Sherman, Texas crash does point out the fact that bridges generally have a design life on the order of 50 or more years so there are likely still many bridges with pre-1965 designed bridge railings. There are likely many un-crash tested bridge railings still on the National Highway System and there are certainly many on the State and local roadway networks that need to be identified and possibly upgraded.

The NTSB has recommended the development of selection criteria for bridge railings and improved bridge railing design for larger vehicles for nearly 30 years. While there has been a great deal of research, design and testing to develop high containment bridge railings the development of selection and location criteria have lagged considerably behind so, while there are now many more higher containment bridge railings available States and designers are still largely left to their own judgment on where and when to use them. While the basic approach set out in the 1989 AASHTO GSBR was a good step forward, its guidance was still largely general and intuitive. The BCAP computer program suggested for use in assessing the cost effectiveness of different performance level bridge railings was found to have problems with its underlying data and it was soon replaced by the more general RSAP program.

Some States are satisfied with TL3 bridge railings in most situations but many other States, particularly more urbanized States, have established TL4 as the minimum acceptable test level bridge railing installed on NHS and/or interstate highways. A few States, generally with more high-volume urban highways, even require that TL5 bridge railings be installed on interstate highways (e.g., New Jersey). Only a few states have established guidelines which define adverse geometry or the percentage of trucks which would require the installation of a higher test level bridge railing. Even fewer states have a policy in place for the systematic retrofitting or replacement of substandard bridge railings, with cost often cited as a major concern when deciding to upgrade to a higher test level for new and retrofit designs. Pedestrian and bicycle accommodations, however, dictate in all reviewed state policies the use of 42 inch bridge railing regardless of costs to reduce the possibility of pedestrians or bicyclists from going over the railing.

There is, therefore, a need to develop more specific recommendations on where different test level bridge railings should be used. While the general guidance provided by the 1989 AASHTO GSBR and the Roadside Design Guide are sound, States and designers need more specific characteristics like what percentage of trucks constitute "large truck traffic," or what traffic volume might be considered "high volume," or what degree of curvature constitutes a "sharp curve." Providing these more specific answers is one of the objectives of this research project.

RESULTS OF SURVEY OF PRACTICE

Introduction

A survey of practitioners was distributed via e-mail to about 2,800 roadside safety researchers, bridge engineers, DOT engineers, and highway design consultants both within the US and abroad. This survey was conducted to determine the current policies for deciding which test level bridge railing to use in particular situations and to obtain data which could be used in this research. The distribution list was compiled from the AASHTO Subcommittee for Bridges and Structures mailing list, the TRB AFB20 and AFB20 subcommittees' mailing lists, the ITE database, AASHTO-ARTBA-AGC TF13 mailing list, ATSSA training course participant list, and from a list of people who have purchased the Roadside Design Guide from AASHTO.

The survey was assembled and made available using the on-line tool surveymonkey.com (i.e., www.surveymonkey.com). The survey had several purposes including:

- What is the default bridge railing test level used in each State or country?
- Are there specific warrants for using a different test level than the default?
- If there are not specific warrants, are there informal guidelines or criteria for deciding when to use a different performance bridge railing?
- Are construction or repair cost data available for bridge railings?
- Can the bridge inventory be linked to the States crash data?

Approximately 54 people started the survey and 45 people completed it resulting in a completion rate of 83 percent. The remaining recipients that did not respond in any way are presumed to know nothing about the selection of bridge railings or are not active in the roadside safety aspects of bridge design. The survey asked a variety of questions about bridge rail selection guidelines in use, bridge railing inventories and crash databases. The following sections discuss each question and summarize the responses.

Survey Questions

Question 1: Please provide the following optional information about yourself.

Respondents were asked to provide contact information. Approximately 90 percent of the respondents provided this information. Respondents represent a variety of countries including the United States, England, China, and New Zealand. Representatives from thirty-two states responded (listed below), however, not all of these respondents represent a state Department of Transportation (DOT). Many are consultants or researchers who have exposure to local and regional design guidelines. Additionally, some respondents represent county level engineering departments responsible for their own bridge rails.

1. Alabama
2. Florida
3. Georgia
4. Hawaii
5. Illinois
6. Indiana
7. Iowa
8. Kansas
9. Kentucky
10. Louisiana
11. Maryland
12. Massachusetts
13. Michigan
14. Minnesota
15. Mississippi
16. Nebraska
17. Nevada
18. New Jersey
19. New York
20. North Dakota
21. Ohio
22. Oregon
23. Pennsylvania
24. Rhode Island
25. South Carolina
26. South Dakota
27. Texas
28. Utah
29. Virginia
30. Washington
31. Wisconsin
32. Wyoming

In summary, this wide variety of respondents provided a good cross-section regarding the type of data available for this study as well the practices currently in use both nationally and internationally.

Question 2: What are the approximate percentages of TL2, TL3, TL4, TL5, and TL6 bridge rails you have in your jurisdiction?

Respondents were asked to provide specific information about the distribution of bridge rails within their jurisdictions. The test levels referred to in this question are Report 350 test levels. The approximate percentage, not mileage, of the rails was obtained from thirty-five of the people who took this survey. An additional five participants answered "unknown" while many others skipped the question altogether presumably because they could not provide an estimate. Figure 37graphically displays the distribution of results. The horizontal axis is the range of possible percentages from the respondents. The vertical axis is a ratio of the responses per total respondents to the question which fall into each category. There appears to be a small percentage of TL5 bridge rails currently installed, while other test levels appear to vary widely. Only one respondent indicated that TL6 bridge rails are installed, therefore, the TL6 rails were not included in Figure 37.

Another way to look at these results is to consider the average value for each category. Table 22 provides the average values of the responses. Again, there appear to be few TL5 bridge rails currently in-service, however, TL2 through TL4 barriers appear to have wide usage rates.

Table 22. Question 2 Average Values.

TL2	20%
TL3	32%
TL4	51%
TL5	6%

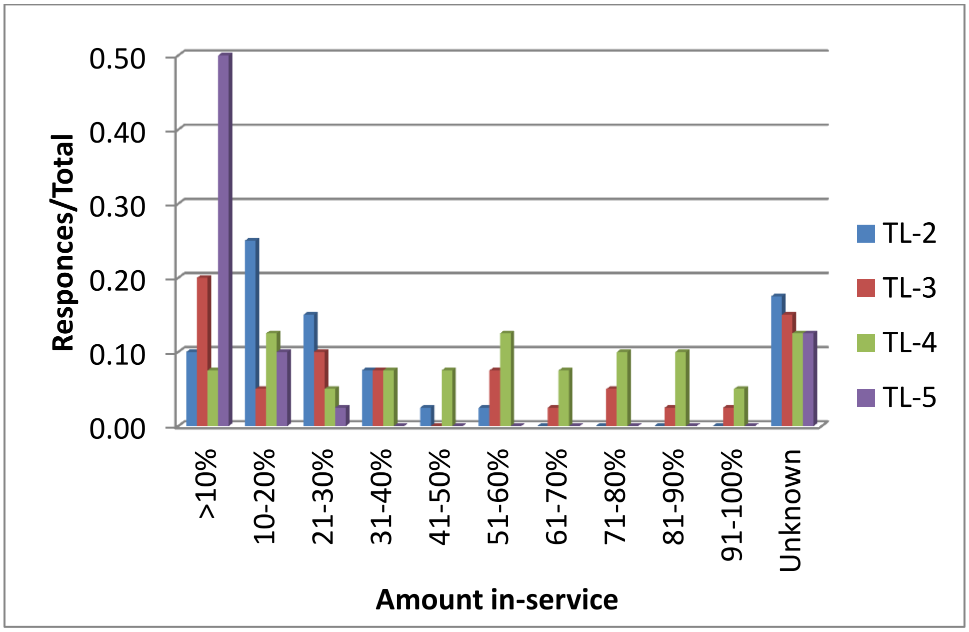


Figure 37. Distribution of Reponses to Question 2.

With regards to TL4 bridge rails, respondents noted that "65% of new LRFD designed bridges" use TL4 bridge rails or that "all new construction since early 80's uses TL4." One respondent also pointed out the disparity between State and Federal routes verses County and Town routes when noting that "95% State and Federal routes; 10% County/Township" are using TL4 bridge rails. It is important to also keep in mind, as a respondent pointed out, that some jurisdictions have "60% TL2 or less timber rail from 1960 or earlier."

In summary, there is a wide range of different bridge railing test levels in use. Not surprisingly, the most common bridge railings installed appear to be TL3 and TL4. TL4 bridge railings appear to be the most common at least on newer construction and there are a few higher-level bridge railings as well (i.e., TL5).

Question 3: As part of this work, the research team is collecting in-service crash records for bridge railings. If you or your agency has data available to help with the development of these guidelines, please list the best way to contact you and the nature of the data in the box below.

Four survey respondents listed contact information and suggested the research team contact them directly. The information gained from these individuals was included earlier in the Literature Review.

Question 4: Are you aware of a bridge inventory which can be linked to State crash data?

The respondents are not aware of any bridge inventories which are linked to State crash records. One respondent acknowledged that it may be possible but noted that "…it is a difficult process. It requires a tedious manual effort of overlapping two data bases of data. Unfortunately we don't have the personnel available to provide this information in a timely manner."

Question 5: Has your State sponsored any in-service performance studies of bridge rails or median barriers?

While not a bridge rail study *per se*, the state of Washington conducted an in-service review of cable median barriers and concrete safety shape median barriers. The concrete safety shape results of that study were useful in postulating the effectiveness of similar concrete safety shaped bridge railings. A similar situation appeared for New Jersey where there is a study of concrete median barriers which provided some insight on similar concrete bridge railings. The survey respondent noted these documents can be found here:

- Cable median barriers
  (http://www.wsdot.wa.gov/Projects/CableBarrier/Report2009.htm)
- Jersey shape median barriers
  (http://www.wsdot.wa.gov/publications/fulltext/design/RoadsideSafety/TRB_Report.pdf)

A survey respondent suggested the research team review documents found at http://www.highways.gov.uk/business/14008.aspx. These documents are research reports for a study of "Whole Life Cost-Benefit Analysis for Median Safety Barriers" where the costs for cable barrier and concrete barrier penetration crashes, the costs for the relocation of services when concrete barrier is installed, and the costs for traffic management were reviewed. Again, the study is not specifically about bridge railing but was used to obtain some useful information about similar roadside barriers extrapolated to bridge railings.

One survey respondent suggested the research team review a Michigan study on the performance of concrete railings and barriers at http://www.michigan.gov/documents/mdot/MDOT_Research_Report_R1498_207581_7.pdf. Michigan conducted a study of the premature deterioration of Michigan's concrete railings and barriers. The study found that material specifications, construction methods and maintenance practices all contributed to the observed deterioration. In addition to the reports listed above, one respondent suggested that the research team make direct contact to obtain information.

In summary, a few in-service studies of median barriers or bridge rails were reviewed in detail and have been summarized in the Literature Review section of this report.

Question 6: Are you aware of construction or repair costs available for bridge rails?

Many respondents suggested that bridge rail repair and retrofit costs range from "$50 to $300 per linear foot based on the extent of repair/retrofit" needed and the type of railing under repair.

New construction costs in Massachusetts, for example, were reported as:

- "S3-TL4 metal railing: $320/FT;
- TL4 concrete barriers: $150/FT; and
- TL 5 concrete barrier: $200/FT."

These costs were "calculated from the price guidelines in the MassDOT Bridge Manual that can be downloaded from the following link: http://www.mhd.state.ma.us/default.asp?pgid=content/bridgeman_new02&sid=about."

Respondents provided these additional websites and directions for locating bridge rail construction costs:

- http://www.txdot.gov/business/avgd.htm
- Historical construction costs for some FDOT standard bridge traffic railings are available here under Items 0460-71-1 and 0521-5-x: ftp://ftp.dot.state.fl.us/LTS/CO/Estimates/12MonthsMoving.pdf
- http://www.tdot.state.tn.us/construction/Average%20Unit%20Prices/aup_2010.pdf
- Once in the Indiana DOT website, in http://www.in.gov/dot/div/contracts/pay/ - click on English Unit Price Summary, then scroll to 706 in column A. Records identified as 706 are the bridge-railing pay items.

In summary, repair and retrofit costs range from $50 to $300/LF while new construction ranges from $150 to 320/LF. It is assumed there are large discrepancies by region and bridge rail test level, as well as bridge rail design. Construction costs are further reviewed and discussed later in this report.

Question 7: In the conduct of your work, have you encountered guidelines for the placement of particular test level bridge rails or median barriers under certain situations?

Reponses varied considerably to this question. One respondent stated that "PennDOT uses a TL5 barrier except in limited situations where sight distance is an issue. For these situations a 32 inch barrier is used." Conversely, another respondent said that "WYDOT uses TL3 rails on all bridges, except on I-80 and in areas of high truck traffic, then TL4 railing is used." Other respondents indicated that Alabama specifies a minimum of TL4 railing; WSDOT requires "TL5 at T-intersections on a bridge or structure or when the barrier is on the outside curve of a structure with radius of curvature less than 500 ft." WSDOT also requires TL5 "where approach speeds are 50 mph or greater." "Some standards call for the use of TL5 bridge rails for horizontal curved bridge with high speeds while allowing for TL3 bridge rails to accommodate historic bridge needs."

Many States rely on the 1989 AASHTO Guide Specifications for Bridge Railing in combination with the AASHTO LRFD Bridge Design Specifications, AASHTO Roadside Design Guide, and the AASHTO Task Force 13 barrier guide. Some States, including Utah and New Mexico, are in the process of updating design standards to include guidelines for the selection of bridge rails and median barriers. Maryland has a draft Policy and Procedure Memorandum (D-78-16(4) Barrier Railing Systems on New or Rehabilitated Structures) for selecting bridge railings.

Internationally, the Chinese standard is issued by Ministry of Transport. In the UK, the specification is TD19/06, and New Zealand follows NCHRP 350 guidelines and at times references EN1317.

In summary, there does not appear to be any consensus about when to use what type of bridge railing, however, there are some existing guidelines.

Question 8: Does your organization have a published set of guidelines for selecting bridge rails?

Several survey respondents provided references to specific design guidelines which were reviewed and summarized in the literature review. These references are shown here:

- FDOT Structures Design Guidelines, Section 6.7: http://www.dot.state.fl.us/Structures/StructuresManual/CurrentRelease/StructuresManual.shtm
- MNDOT Bridge Design Manual: http://www.dot.state.mn.us/bridge/manuals/LRFD/index.html
- NJDOT Design Manual for Bridges and Structures, 5th Edition, Section 23.: http://www.state.nj.us/transportation/eng/documents/BSDM

- Rhode Island Bridge Engineering Design Guides http://www.dot.ri.gov/engineering/guides/index.asp
- Michigan Bridge Design Manual: http://www.michigan.gov/mdot/0,1607,7-151-9622---,00.html
- Chapter 13 of Design Manual Part 4. ftp://ftp.dot.state.pa.us/public/PubsForms/Publications/PUB%2015M.pdf
- Chapter 3 of Part I of the MassDOT Bridge Manual that you can download from the following link: http://www.mhd.state.ma.us/default.asp?pgid=content/bridgeman_new02&sid=about
- NYSDOT Bridge Manual: https://www.nysdot.gov/divisions/engineering/structures/repository/manuals/brman-usc/Section_6_US_2010.pdf
- http://www.dot.state.oh.us/Divisions/HighwayOps/Structures/standard/Bridges/test/railing%20selection%20procedure.pdf
- NCDOT Design Manual, Section 6.2.4: http://www.ncdot.org/doh/preconstruct/highway/structur/designmanual/lrfd/LRFDManual(December2010).pdf
- http://www.dot.nd.gov/manuals/design/designmanual/designmanual.htm
- http://www.dot.state.mn.us/bridge/manuals/LRFD/index.html
- SCDOT Bridge Design Manual: http://www.scdot.org/doing/bridge/06design_manual.shtml
- Illinois Bridge Manual Section 2.3.6.1.7.: http://www.dot.il.gov/bridges/brmanuals.html
- INDOT Design Manual: http://www.in.gov/dot/div/contracts/standards/dm/english/index.html, click on "Structural Design", then BRIDGE DECKS. Scroll down the bookmark to 61-6.0 BRIDGE RAILING, then click on it. The Manual copy then appears on the right.
- NDOT Structures Manual, Chapter 16, Section 16.5.1.2.: http://www.nevadadot.com/divisions/011/
- WSDOT BDM, Chapter 10. Search the WSDOT website for the most recent electronic version.
- Virginia Department of Transportation Structure and Bridge Division Manuals/Manual of the Structure and Bridge Division, Volume V-Part 2, Chapter 25

Question 9: Which guidelines does your organization use for selecting/specifying bridge rail crash test performance?

Approximately two-thirds of the respondents indicated that NCHRP Report 350 is used for specifying crash test performance of bridge railings, however, many respondents indicated that the organizations they represent are transitioning to MASH. Participants also noted the Chinese standard of transport JT/T F83-2004 and EN1317 are used for specifying bridge rail crash test performance internationally.

Question 10: Which criteria do you use in the rail selection process?

Survey respondents were asked which criteria apply when selecting bridge rails. Respondents were given several choices and allowed to select as many choices as applicable. Choices included:

- Engineering Judgment,
- ADT,
- Percent Trucks,
- Posted Speed Limit,
- Design Speed,
- Number of Lanes,
- Accident data,
- Cost/benefit,
- Roadway type,
- Horizontal alignment,
- Vertical alignment,
- Land use around the structure, and
- Other.

Design speed, traffic characteristics (i.e., volume and truck percentage), and engineering judgment have the most impact on the selection of bridge rails while the roadway type, horizontal alignment, and land use also appear to significantly impact selection. All of the responses are displayed in Figure 38.

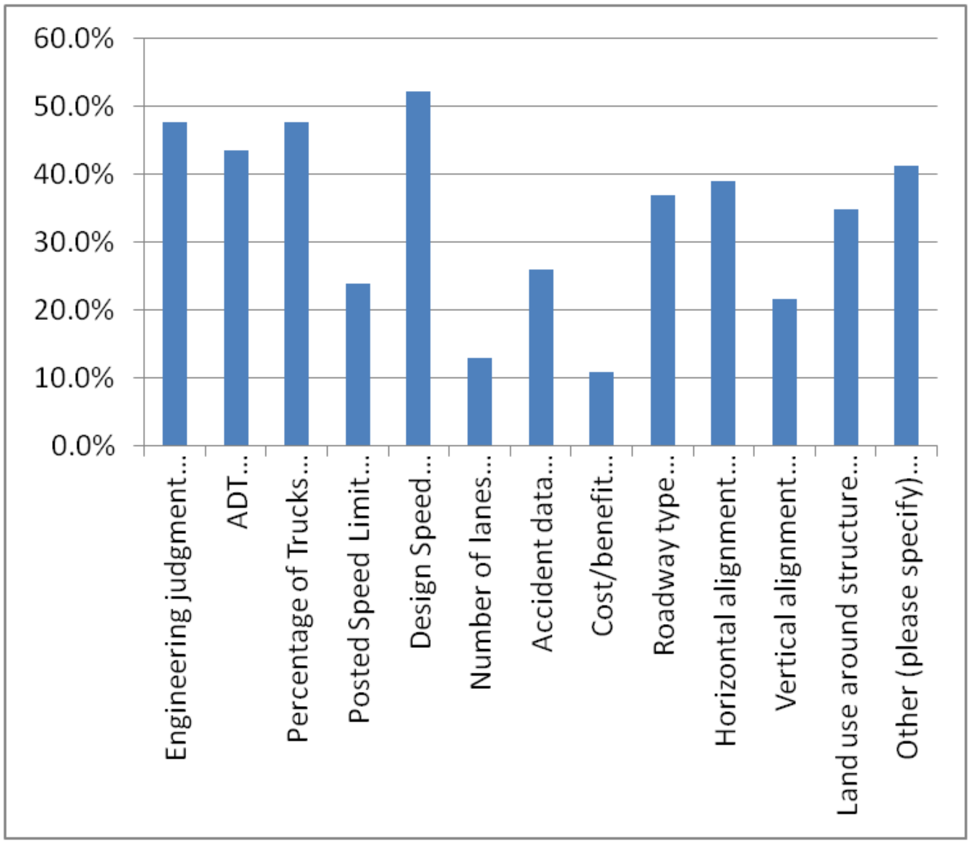


Figure 38. Distribution of Question 10 Results.

Survey respondents noted that accommodations for pedestrians, bicycles, flood waters, snow removal and scenic views also play rolls in the selection of bridge rails. Other considerations range from shoulder width and sight distance to funding source (i.e., local, state, federal, etc.) Often times the weight and maintenance costs of the rail are concerns. "Aluminum rails are being stolen at an alarmingly high rate, so the state of Iowa is proactive about replacing them with the lightest functional rail available."

One survey respondent noted "all of the above criteria are considered, however, there are not any published guidelines that cover all of the possible combinations …. A formula would be nice for determining what test level to use."

Question 11: Does your organization have a policy for identifying and/or retrofitting substandard bridge rails?

Approximately 40 percent of the respondents indicated the organization they represent does not have retrofitting policies. Many of the 60 percent of respondents, who do have policies, have informal or unwritten policies. Many States inspect bridge railing during bridge inspections. Substandard rails are identified based on age and height in some cases or by type and visual inspection.

Respondents noted that when a highway safety or resurfacing project is under construction in the area, “any structures within the limits of work are evaluated for the parapet/railing type and condition, as well as the transition from the roadway approach barrier to the bridge parapet. Deficient transitions are typically brought up to standards within the safety/resurfacing contract. Deficient bridge barriers are noted and included in a regionalized list of deficient bridge barriers. Periodic area-wide contracts are procured to upgrade deficient bridge barriers within a region.” Other respondents note that rails are only reviewed “per federal funding criteria, when an existing bridge is being rehabilitated.”

Respondents suggested a review of policies at these locations:

- Minnesota Bridge Preservation, Improvement and Replacement Guidelines: http://www.dot.state.mn.us/bridge/documentsformslinks/techmemos/06-10-b-01.pdf. A more current version should be available soon.
- ODOT Bridge Design Manual, Section 304.1: http://www.dot.state.oh.us/Divisions/HighwayOps/Structures/standard/Bridges/BDM/BDM2004/BDM2004_10-15-10.pdf
- Indiana Design Manual: http://www.in.gov/dot/div/contracts/standards/dm/english/index.html - click on "BRIDGE REHABILIATION". 72-3.01(03) Rehabilitation Techniques. Table BD-7. Also, 72-7.02(04) Bridge Railing.
- Illinois Bridge Design Manual: http://www.dot.il.gov/desenv/demanuals.html
- WSDOT Bridge Design, Section 10.4. Search for "BDM" on the WSDOT home page.

It appears very few organizations have formal policies for identifying substandard bridge rail.

Question 12: What testing has been done to support the retrofitting of substandard bridge rail policy?

Respondents indicate that retrofitting policy is based on materials found in NCHRP Report 350, the 2005 FHWA/CALTRANS Bridge Rail Guide, and historic crash tests. [FHWA05] Many respondents indicated that no additional testing has been done in support of the policies, which is not surprising given the lack of formal policies.

Question 13: Does your organization have a default bridge rail crash performance Test Level installed/specified in the absence of other controlling criteria?

Approximately 65 percent of the respondents indicate that the organization they represent have minimum requirements for bridge rail crash performance. Two respondents went on to explain this minimum is TL3 while twelve respondents explained the minimum is TL4. Three respondents indicated that TL5 railings are used exclusively on interstate highways.

Question 14: What is the default bridge rail?

Respondents indicated a variety of default rail choices. Many defaults including 32" concrete barriers such as the New Jersey shape, the F-Shape and the Single Sloped shape. Some respondents report the default use of 42" single sloped concrete on the Interstate and US Route systems. Other default rails are shown here:

- TL3 steel two tube railing,
- Kansas corral rail,
- Oregon Rail,
- 34" Single Slope,
- Side mounted R-34 railing,
- S3-TL4 where a see-through railing is needed,
- Usually the 4-rail system, or
- EN1317, N2 or H4a.

In summary, responses suggest a variety of default rails currently in use. Over 65 percent of respondents, in fact, indicate that default rails are suggested in their region.

Question 15: Are there any reasons for not installing the default bridge rail?

Not surprisingly, over 75 percent of the respondents indicated that there are often reasons for not installing the default bridge rail. In some cases, the standard rails do not meet the needs of the situation. For example, the rails may not attach to some types of bridges with adequate height, rails may not combine with sound barriers or rails may not transition well to existing rails. Respondents noted that aesthetics, the need for a higher or lower test level, vertical and/or horizontal alignment and sight distance issues may all lead to installing bridge rails other than the default.

Respondents also noted that often times a super-elevated horizontal curve, a high design speed, significant truck traffic or high crash rates may lead to the installation of rails other than the default. Sometimes it is more cost effective to use different construction methods (e.g., extrude the railing). Dead load may be an issue as well, especially for re-decking projects. In historic areas or areas using context sensitive design principles, some lower test level rails are installed for aesthetic reasons.

In summary, many highway design elements are factors when choosing a bridge rail other than the default.

Summary

It appears there are few in-service studies of median barriers or bridge rails. The respondents were not aware of any bridge inventories which are linked to State crash records. Only four crash databases were located as part of this survey. Survey respondents noted the difficulty in linking crash records with state databases.

There does not appear to be any consensus about when to use what type of bridge railing, however there are some existing guidelines both formal and informal. Several survey respondents provided references to specific design guidelines. Approximately 65 percent of the respondents indicate that the organizations they represent have minimum requirements for bridge rail crash performance and/or default bridge railings. Many highway design elements are factors when choosing a bridge rail other than the default, however these design decisions and polices appear to lack formality. One survey respondent noted "A formula would be nice for determining what test level to use."

Few organizations have formal policies for identifying substandard bridge railings, however, retrofitting policies are based on materials found in NCHRP Report 350, the 2005 Bridge Rail Guide, and historic crash tests. Many respondents indicated that no additional testing has been done in support of the policies.

The wide variety of respondents represented a good cross-section of the current practices both nationally and internationally. With bridge rail repair and retrofit costs ranging from $50 to $300/LF while new construction ranging from $150 to 320/LF, the time for formal national guidelines which examine the appropriate installation of different test level barriers under different scenarios has come.

DEVELOPMENT OF BRIDGE RAIL SELECTION GUIDELINES

The available analysis methods and software products presented in the literature review assess the probability of a roadside feature being struck, the severity of the crash if it has occurred and the resulting crash costs through conditional probabilities then perform a benefit-cost analysis of roadside design alternatives to determine the most cost-effective design. RSAPv3 has the added capability of tabulating crash costs such that the lifetime risk of a specified crash severity can be calculated independent of the direct costs (i.e., construction, maintenance and repair costs). The risk analysis and cost/benefit analysis capabilities of RSAPv3 were employed in the development of the selection guidelines. RSAPv3 uses this conditional probability model [Ray12]:

$$E(CC)_{\mathrm{N,M}} = \mathrm{ADT} \cdot \mathrm{L_N} \cdot P(Encr) \cdot P(Cr|Encr) \cdot P(Sev|Cr) \cdot E(CC_s|Sev_s)$$

where:

$E(CC)_{\mathrm{N,M}}$ =	Expected annual crash cost on segment N for alternative M,
ADT =	Average Daily Traffic in vehicles/day,
$\mathrm{L_N}$ =	Length of segment N in miles,
$P(Encr)$ =	The probability a vehicle will encroachment on the segment,
$P(Cr\|Encr)$ =	The probability a crash will occur on the segment given that an encroachment has occurred,
$P(Sev_s\|Cr)$ =	The probability that a crash of severity *s* occurs given that a crash has occurred and
$E(CC_s\|Sev_s)$=	The expected crash cost of a crash of severity *s* in dollars.

An RSAPv3 analysis is composed of four major steps for assessing each alternative:

- Encroachment Probability,
- Crash Prediction,
- Severity Prediction, and
- Benefit-Cost and/or Risk Analysis.

Some improvements were made to RSAPv3 in this project to specifically address the challenges of developing bridge rail selection guidelines. These upgrades are discussed below.

Crash Data

Unfortunately, the literature review and survey did not uncover much in the way of in-service studies or even crash studies of bridge railings so it was necessary to look for other sources of data for bridge railing performance to use in the development of the Selection Guidelines. There are some existing databases like the NCHRP22-08 and FHWA Narrow Bridge databases discussed earlier but these are now very old and, at least in the case of the NCHRP 22-08 Texas data, have been determined to have serious coding problems. In any case, a great deal has changed in the types of bridge railings that are available today in comparison with the early 1990s as reflected in the NCHRP 22-08 report.[Mak94]

Crash data was collected and used to populate the crash severity module of RSAPv3 for several concrete bridge railings and similarly shaped median barriers for use in this project. Crashes with concrete median barriers were reviewed to determine the severity distribution for striking the bridge railings of similar shape. Concrete median barriers were chosen because there are more miles of median barrier installed than bridge railing which maximizes the amount of crash data that is available for modeling the severity of crashes with typical median barrier and bridge railing shapes. Most concrete median barriers and concrete bridge railings use essentially the same shapes (i.e., New Jersey shape, F shape, vertical wall, constant or single slope, etc); therefore, these roadside devices are expected to perform similarly with respect to crash severity.

Crashes with bridge rails were analyzed to determine the severity distribution of crashes where the bridge railing was penetrated. All types of bridge rails were considered in this analysis because the analysis was focused on the result of penetration, not the probability of penetration. The type of bridge rail would not impact the outcome after penetration since the outcome is a function of the land use characteristics around and under the bridge (i.e., presence of other transportation facilities, urban or rural areas, etc.). Crashes with embankments and water hazards were also considered as a reference point. The following sections briefly describe the data used in these analyses. The computational steps and results of the analyses are discussed throughout this report in the relevant sections.

New Jersey Median Barrier

Crash records for the New Jersey Turnpike were requested for 2003 through 2009 from Rutgers University. Rutgers maintains a database of crashes throughout New Jersey which are linked to road geometrics.[Plan4S11] The ADT and percent of trucks for the New Jersey Turnpike were obtained directly from the New Jersey Turnpike Authority (NJTA). A 105-mile long section of the New Jersey Turnpike was selected for study where a TL5 concrete safety shape median barrier is used exclusively and continuously for the entire length of the highway. [NJTA1] The speed limits on this 105-mile long section were either 55 or 65 mi/hr. A total of 1,816 crashes within the 65 mi/hr zone and 241 crashes within the 55 mi/hr zone were obtained and reviewed. The severity distributions were calculated and adjusted for unreported crashes and the $EFCCR_{65}$ was determined (see the section entitled "Severity" on page 135 for a discussion of the $EFCCR_{65}$). The distributions for each speed are presented in Table 23. The penetrations and rollovers percentages were also determined and are presented in Table 24. Narratives of the crashes for which the vehicles appear to have penetrated or rolled over the barrier were requested and subsequently reviewed to verify that a PRV had occurred.

Table 23. New Jersey Turnpike Crash Severity Distribution.

Barrier	K	A	B	C	PDO/UNK	Total
Contained, Stopped or Redirected on 55 mi/hr Segments						
No.	0	1	12	35	193	241
%	0.00	0.41	4.98	14.52	80.08	100
Contained, Stopped or Redirected on 65 mi/hr Segments						
No.	0	11	103	307	1395	1816
%	0.00	0.61	5.67	16.91	76.82	100

As shown in Table 24, two instances of a barrier PRV in the 55 mi/hr zone occurred. One of these resulted in a possible injury (i.e., C), while the other resulted in property damage only crash. There were 11 instances of barrier PRV in the 65 mi/hr zone. Two of these resulted in visible injuries, one in a possible injury, five in property damage only crashes, and the injury level was unknown for three of the cases.

Table 24. NJTA After Barrier Contact Behavior.

Behavior	55 mi/hr #	65 mi/hr #
Contained/Redirected	241	1,816
PRV	2	11
Rollover After Redirection	5	41

The number of instances where the vehicle rolled over after being redirected by the barrier were also collected and are shown in Table 24 for both of the speed zones. In the 55 mi/hr zone, five instances of redirection rollovers occurred; two had visible injuries, two had possible injuries, and one resulted in property damage only. In the 65 mi/hr zone, 41 instances occurred; 1 fatality, 1 incapacitating injury, 19 visible injuries, 11 possible injuries, 4 resulted in property damage only, and 5 cases where the injury level was unknown.

Massachusetts Median Barrier

The Massachusetts DOT crash database was also examined for 2006-2009 to identify median barrier collisions on specific sections of roadways where median barriers were recently constructed (i.e., within the past five or six years). A subsequent field review was conducted to isolate sections of roadway where 32-inch tall and 42-inch tall concrete F-shape median barriers exist absent of other types of barriers. This field review was conducted to eliminate the possibility of reviewing crash records where the reporter may have confused the type of barrier struck. After this review, 154 crashes with 32-inch barrier and 34 crashes with 42-inch barrier were identified. All of these crashes occurred on roads with posted speed limits of either 55 or 65 mi/hr. The severity distribution was determined and the results and distributions are shown in Table 25. The percentages of penetrations and rollovers were determined and confirmed using available narratives of the police reports. These percentages are presented in Table 26.

From Table 26 it can be seen that the 32 inch F-shape barrier had two reported instances where the vehicle penetrated, rolled, or vaulted over the barrier in the 55 mi/hr speed zones. Both of these crashes resulted in non-incapacitating injuries. This same barrier had six instances of PRV failure in the 65 mi/hr zone as well, two of which were non-incapacitating injuries and four resulted in property damage only. The 42 inch F-shape barrier had two instances of PRV failure in the 55 mi/hr zone (which was the only zone the 42 inch F-shape was installed), and both instances resulted in property damage only.

Also in Table 26 is the rollover after redirection information for the different barriers within the different speed zones. No rollover after redirection cases were reported for the 32 inch F-shape barrier within the 55 mi/hr speed zones, but five were reported in the 65 mi/hr zones. Of these five crashes, three resulted in non-incapacitating injuries, one in property damage only, and one crash had an unknown level of injury. Only one occurrence of a rollover after redirection was reported for the 42 inch F-shape barrier in the 55 mi/hr zones (again, this was the only speed zone where crashes were analyzed for this F-shape barrier). Unfortunately, this single rollover resulted in a fatality.

Table 25. Massachusetts Crash Severity Distribution.

Barrier	K	A	B	C	PDO/UNK	Total
Contained, Stopped or Redirected on 55 mi/hr Segments, 32" F-Shape						
No.	0	0	4	4	14	22
%	0.00	0.00	18.18	18.18	63.63	100
Contained, Stopped or Redirected on 65 mi/hr Segments, 32" F-Shape						
No.	3	4	36	17	72	132
%	2.27	3.03	27.27	12.88	54.55	100
Contained, Stopped or Redirected on 55 mi/hr Segments, 42" F-Shape						
No.	0	0	6	4	24	34
%	0.00	0.00	17.65	11.76	70.59	100

Table 26. Massachusetts After Barrier Contact Behavior.

Behavior	32" @ 55 mi/hr	32" @ 65 mi/hr	42" @ 55 mi/hr
	#	#	#
Contained/Redirected	22	132	34
PRV	2	6	2
Rollover After Redirection	0	5	1

Washington State Median Barrier

The Washington State crash data was examined for I-90 and I-5 with posted speed limits of 60 mi/hr where 32-inch New Jersey safety shape and 34-inch single-slope concrete median barriers were used. The severity distributions of 549 cases involving 32-inch safety shape barriers and 178 cases involving single-slope barriers were calculated and can be seen in Table 27. The behavior after contact was determined for both barriers and is presented in Table 28.

Table 27. Washington State Crash Severity Distribution.

Barrier	K	A	B	C	PDO/UNK	Total
Contained, Stopped or Redirected on 60 mi/hr Segments, 32" Safety Shape						
No.	2	4	62	112	369	549
%	0.36	0.73	11.29	20.40	67.21	100
Contained, Stopped or Redirected on 60 mi/hr Segments, 34" Single Slope						
No.	0	3	20	28	127	178
%	0.00	1.69	11.24	15.73	71.35	100

Table 28. Washington State After Barrier Contact Behavior.

Behavior	32" Safety Shape #	34" Single Slope #
Contained/Redirected	549	178
PRV	3	1
Rollover After Redirection	14	6

Table 28 shows the reported crashes that resulted in a penetration, roll or vault over the barrier (PRV). For the 32 inch safety shape barrier, three of these crashes occurred. One of these crashes resulted in a non-incapacitating injury, while the other two resulted in property damage only. For the 34” single slope barrier, the only crash of this type resulted in property damage only.

Table 28 also shows the number of reported crashes that resulted in a rollover after being redirected by the two barrier types. The 32 inch safety shape barrier had 14 of these crashes; including one that resulted in a fatality. Six of the remaining 13 crashes resulted in non-incapacitating injuries (level B), three resulted in possible injuries (level C), and four resulted in property damage only. The 34 inch single slope barrier had six crashes where the vehicle rolled over after being redirected by the barrier. Two of these crashes resulted in non-incapacitating injuries, one resulted in a possible injury, and three resulted in property damage only.

Pennsylvania Bridge Railing

The Pennsylvania Department of Transportation (PennDOT) requires "bridge railings that meet the requirements of Test Level 5 (TL5) of NCHRP Report 350, unless another test level is authorized by the District Executive." [PennDOT11] PennDOT generally specifies a 42-inch concrete F-shape barrier as the TL5 railing, however, other PennDOT adopted railings may also be used. A TL4 32-inch concrete F-shape barrier is also a common barrier used and was also considered in this research.

Crash records were reviewed from 2006 to 2010 for bridge rail crashes on interstates highways. Traffic volumes for the interstates and the roads which crossed under the interstates were found online.[PA11] Unfortunately, the percentage of trucks in the traffic was not available. The environmental features surrounding each bridge were reviewed using Google Earth.[Google11]

Table 30 shows the reported PRV events for the two heights of the F-shape bridge rail in both the 55 and 65 mi/hr speed limit zones. For the 32" F-shape bridge rail in the 55 mi/hr zone, two PRVs occurred. The first of these involved a tractor-trailer where the trailer rolled over and stayed on the bridge but the tractor broke through the barrier and dropped off the bridge. The tractor fell approximately 90 ft and landed on its passenger side on a small island in the middle of Maiden Creek, resulting in a possible injury. Witnesses say the truck was travelling at or around the speed limit at the time of the crash.

The second 32 inch bridge rail crash that occurred in the 55 mi/hr zone involved a passenger car that vaulted over the rail and fell approximately 80 ft and came to rest on its roof in a wooded area next to a river. This crash resulted in property damage only.

The same bridge rail in the 65 mi/hr zone experienced five PRVs. The first of these involved a tractor-trailer truck where the tractor portion stayed in the traveled way of the highway, but the trailer portion broke through the bridge rail and ended up hanging over the bridge but did not fall off. This resulted in an incapacitating injury. This bridge crosses Pulaski Mercer Road (State Route 468), which experiences a 400 vehicle per day traffic volume.

The second 32-inch bridge rail PRV crash that occurred in the 65 mi/hr zone involved a passenger car which mounted the railing and rode on top of the rail for 30 ft before falling off the bridge and dropping 60 ft. It came to rest on its roof, resulting in an incapacitating injury. According to witnesses, this vehicle was travelling somewhere between 50 and 55 mi/hr when the incident occurred, which is below the posted speed limit. It was later discovered that a vehicle defect caused the vehicle to hit the bridge rail. The vehicle landed in an unused area under the bridge; a fortunate occurrence as this bridge spans South Fork Tenmile Creek (Route 188), a 5,000 vehicle per day state route, and a set of railroad tracks.

The third PRV incident for this barrier in the 65 mi/hr speed zone also involved an incapacitating injury. In this crash, the vehicle had rolled over onto its passenger side prior to coming in contact with the bridge rail. As the vehicle hit the bridge rail it rolled over the barrier and landed in a grassy area next to Bullfrog Road (unknown average daily traffic).

Table 29. Pennsylvania Crash Severity Distribution.

Barrier	K	A	B	C	PDO/UNK	Total
Contained, Stopped or Redirected on 55 mi/hr Segments, 32" F-Shape Bridge Rail						
No.	3	1	6	14	33	57
%	5.26	1.75	10.53	24.56	57.89	100
Contained, Stopped or Redirected on 65 mi/hr Segments, 32" F-Shape Bridge Rail						
No.	1	0	7	28	71	107
%	0.93	0.00	6.54	26.17	66.36	100
Contained, Stopped or Redirected on 55 mi/hr Segments, 42" F-Shape Bridge Rail						
No.	1	0	1	3	5	10
%	10.00	0.00	10.00	30.00	50.00	100
Contained, Stopped or Redirected on 65 mi/hr Segments, 42" F-Shape Bridge Rail						
No.	0	0	4	9	33	46
%	0.00	0.00	8.70	19.57	71.74	100

Table 30. Pennsylvania Bridge Rail After Barrier Contact Behavior.

Behavior	32" @ 55 mi/hr #	32" @ 65 mi/hr #	42" @ 55 mi/hr #	42" @ 65 mi/hr #
Contained/Redirected	57	107	10	46
PRV	2	5	0	0
Rollover After Redirection	6	4	3	1

The fourth PRV crash involving the 32 inch F-Shape bridge rail in the 65 mi/hr zone resulted in a non-incapacitating injury. The vehicle struck the bridge rail and then rolled over, landing on the centerline of the road passing below the bridge. This road (Route 374) experiences roughly 1,500 vehicles per day.

The last PRV crash involving the 32 inch F-Shape bridge rail in the 65 mi/hr zone involved a bus filled with 42 passengers and resulted in a single non-incapacitating injury. The bus struck the barrier and rode up onto the top of the barrier and balanced there before falling approximately 100 ft into the Lehigh River below. The bus passengers were evacuated entirely while the bus was balancing on top of the bridge rail.

The 42" F-shape bridge rail had no PRV failures for both the 55 mi/hr and 65 mi/hr speed zones.

The number of incidents for each bridge rail and speed zone where a vehicle rolled over after being redirected by the bridge rail are tabulated in Table 30. Six incidents were reported for the 32 inch rail in the 55 mi/hr speed zone. Two of the incidents resulted in fatalities, two resulted in non-incapacitating injuries and two resulted in possible injuries. For the same size rail in the 65 mi/hr speed zone, four incidents occurred. One of these incidents resulted in a fatality while the remaining three resulted in only possible injuries. The 42-inch bridge rail experienced four cases where the vehicle was reported as rolling over after being redirected; three in the 55 mi/hr zone and one in the 65 mi/hr zone. One of the events in the in the 55 mi/hr zone resulted in a non-incapacitating injury and two resulted in possible injuries. The one reported case where a rollover occurred in the 65 mi/hr zone resulted in a non-incapacitating injury.

Ohio Bridge Railing

Crash data for bridges for bridges in Ohio from 2005 through 2010 includes 4,600 bridge railing crashes. Ohio installs TL3 bridge rail on "all bridge structures on the National Highway System (NHS) or the State System…as defined by NCHRP report 350," effective October 1, 1998. The Twin Steel Tube Bridge Guardrail (Standard Bridge Drawing TST-1-99) should be used for side draining structures, which shall not be used over highways and railroads. "For bridges with heights of 25 feet or more above the lowest groundline or normal water, concrete deflector parapets should be used." [OH11; OH11a] Therefore, the barrier shown in Figure 39 is the TL3 concrete barrier which is typically installed on NHS roadways which cross over highways.

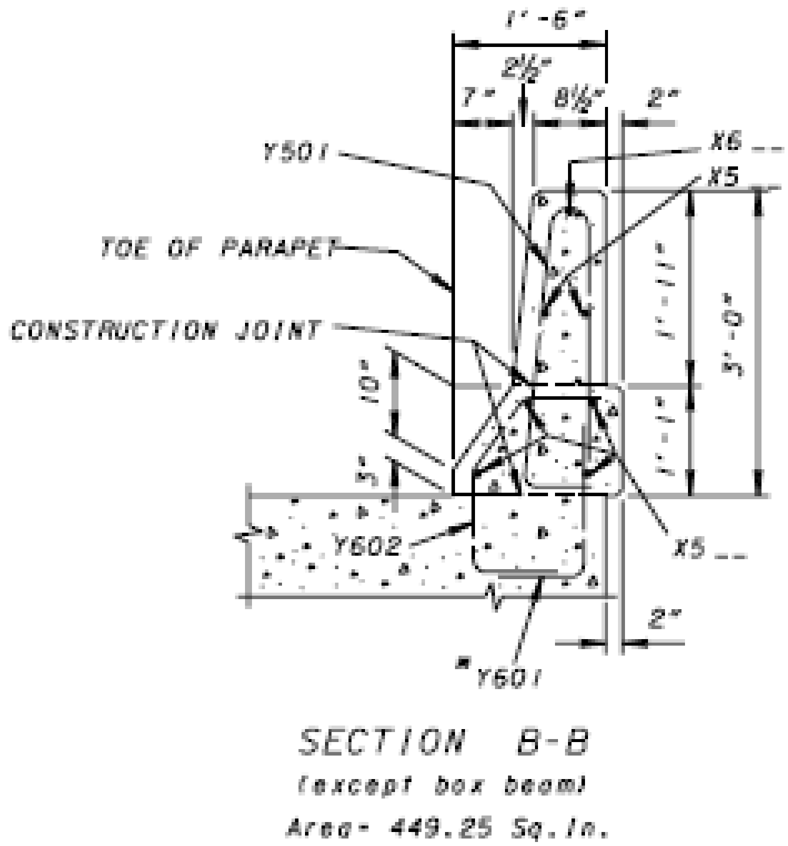


Figure 39. Ohio Standard Drawing BR-1 [OH11a]

The 2005 through 2010 Ohio data indicated that there were 4,560 police reported crashes that involved bridge parapets (i.e., code 28) or bridge rails (i.e., code 29). Vehicles crossed the barrier line 28 times in 4,600 crashes or only in 0.6 percent of the crashes. Of the 28 penetration-rollover-vault (PRV) crashes, none were fatal, four involved A-level injuries (14 percent), 11 involved B-level injuries (39 percent), two involved C-level injuries (7 percent), 10 involved no injuries (36 percent) and one was of unknown severity (4 percent). In three of the cases the vehicle came to rest on a roadway that passed under the bridge and in the remaining 25 cases the vehicle came to rest in or near a body of water. Only one event involved a tractor trailer truck. Table 31 provides a summary of the severity distribution of all reported crashes with bridge rails during the study period and Table 32 summarizes the behavior after the crash.

Table 31. Ohio Crash Severity Distribution.

Posted Speed Limit	K	A	B	C	O	Total
Contained, Stopped or Redirected for 42" Constant Slope						
65	1	1	19	13	83	117
60	0	0	4	2	20	26
55	0	2	4	2	33	41
50	0	0	1	0	4	5
45	0	0	1	1	2	4
Ramp	0	3	2	2	11	18
Total	1	6	31	20	153	211
%	0.47	2.84	14.69	9.48	72.51	100
Contained, Stopped or Redirected for 36" Safety Shape						
65	2	8	63	46	292	411
60	0	3	8	8	34	53
55	0	4	32	27	126	189
50	0	1	1	3	11	16
45	0	0	1	5	26	32
40	0	0	1	0	6	7
35	0	2	2	6	22	32
30	0	0	0	0	0	0
25	0	0	1	1	2	4
Ramp	0	7	23	16	110	156
Total	2	25	132	112	629	900
%	0.22	2.78	14.67	12.44	69.89	100
Contained, Stopped or Redirected for 42" Safety Shape						
65	1	6	38	29	169	243
60	0	8	19	20	78	125
55	0	4	6	9	61	80
50	0	0	2	1	4	7
45	0	0	3	1	8	12
40	0	0	0	1	2	3
35	0	0	2	3	4	9
30	0	1	1	0	1	3
25	0	0	0	0	2	2
Ramp	0	3	16	17	62	98
Total	1	22	87	81	391	582
%	0.17	3.78	14.95	13.92	67.18	100

Table 32. Ohio Bridge Rail After Barrier Contact Behavior.

Behavior	36" #	42" #	Combo rail #	All Rail Crashes
Contained/Redirected	986	873	0	4,449
PRV	5	2	0	29
Rollover After Redirection	28	27	1	82

Nebraska Bridge Rails

The Nebraska DOR crash database was examined for 2007 through 2009 to identify bridge rail collisions on state and local highways, freeways and interstates in Nebraska. The review contained concrete rails and metal rails. These crashes occurred on roads with a variety of posted speed limits. The review of the Nebraska data includes 1,212 crashes on roadways with a variety of posted speed limits. The behavior could be determined for 979 of these crashes. This review included crashes with 29-inch, 34-inch, and 42-inch vertical wall type bridge rails, 32-inch and 42-inch New Jersey shape bridge rails and w-beam type bridge railings. The severity distribution of the crashes which were contained or redirected is shown in Table 33. The behavior of the crashes with these different rails is shown in Table 34.

Table 33. Nebraska Crash Severity Distribution.

PSL	K	A	B	C	O	Total
Contained, Stopped or Redirected for 29" Vertical Wall						
50mph or less	0	0	0	0	0	0
55	0	0	0	0	0	0
60	0	2	2	3	9	16
65	0	0	1	1	2	4
75	0	0	0	0	0	0
Total	0	2	3	4	11	20
%	0.00	10.00	15.00	20.00	55.00	100
Contained, Stopped or Redirected for 34" Vertical Wall						
50mph or less	0	1	3	4	33	41
55	0	4	3	0	20	27
60	3	6	14	28	131	182
65	1	5	9	11	61	87
75	3	4	14	11	102	134
Total	7	20	43	54	347	471
%	1.49	4.25	9.13	11.46	73.67	100
Contained, Stopped or Redirected for 42" Vertical Wall						
50mph or less	0	0	0	0	1	1
55	0	3	3	2	11	19
60	0	0	0	1	1	2
65	0	0	2	2	7	11
75	0	1	0	0	6	7
Total	0	4	5	5	26	40
%	0.00	10.00	12.50	12.50	65.00	100
Contained, Stopped or Redirected for 32" NJ Shape						
50mph or less	0	2	4	2	46	54
55	0	2	1	4	19	26
60	0	2	4	6	36	48
65	0	0	4	2	19	25
75	1	0	1	0	14	16
Total	1	6	14	14	134	169
%	0.59	3.55	8.28	8.28	79.29	100

Table 33. Nebraska Crash Severity Distribution. (CONT'D)

PSL	K	A	B	C	O	Total
Contained, Stopped or Redirected for 42" NJ Shape						
50mph or less	0	0	1	3	4	8
55	0	1	2	6	20	29
60	2	0	3	9	14	28
65	0	0	1	0	1	2
75	0	0	0	0	0	0
Total	2	1	7	18	39	67
%	2.99	1.49	10.45	26.87	58.21	100
Contained, Stopped or Redirected for W-Beam Guardrail						
50mph or less	0	2	0	0	9	11
55	0	1	2	5	11	19
60	2	1	7	7	42	59
65	0	2	5	2	27	36
75	0	0	4	3	30	37
Total	2	6	18	17	119	162
%	1.23	3.70	11.11	10.49	73.46	100

Table 34. Nebraska Bridge Rail After Barrier Contact Behavior.

Behavior	29" Vertical Wall #	34" Vertical Wall #	42" Vertical Wall #	32" NJ Shape #	42" NJ Shape #	W-beam #	All
Contained/Redirected	20	471	67	169	67	162	956
PRV	0	6	0	4	0	14	24
Rollover After Redirection	1	11	0	4	1	10	27

After Penetration Hazards

The Highway Safety Information System (HSIS) is a multistate database that contains crash, roadway inventory, and traffic volume data for a select group of states. [HSIS01] Crash data from Washington State HSIS data for embankments and water hazards were reviewed. Currently, information is available for embankments in 55 and 70 mi/hr speed zones and water hazards in 55 mi/hr speed zones. Table 35 shows the severity distribution of crashes where a bridge rail was penetrated and the vehicle entered the embankment or water hazard.

Table 35. Severity Distributions for After Penetration Hazards

Hazard	K	A	B	C	PDO/UNK	Total
Embankment in 55 mi/hr Speed Zones						
No.	7	10	42	33	93	185
%	3.78	5.41	22.70	17.84	50.27	100.00
Embankment in 70 mi/hr Speed Zones						
No.	3	3	16	5	25	52
%	5.77	5.77	30.77	9.62	48.08	100
Water in 55 mi/hr Speed Zones						
No.	1	2	6	4	37	50
%	2.00	4.00	12.00	8.00	74.00	100

Summary of Crash Data

This crash data has been used throughout this research effort to develop the severity models used within RSAPv3 and to validate the results obtained. The individual analyses conducted with this data are discussed in the relevant sections.

Encroachment

The probability that a vehicle will encroach (i.e., vehicle leaving the road) on a segment is the first event considered in a series of conditional events evaluated in RSAPv3. These conditional events include: the encroachment probability, the probability of crash given an encroachment, the severity of the crash if an object is struck and the cost of the entire crash sequence. The probability of an encroachment has been the focus of several studies in the last forty years, however very little successful data collection on the frequency of encroachments has been accomplished. Data collected by Cooper and by Hutchinson and Kennedy have received much attention, but there are few alternate sources of encroachment data. [Cooper82; Hutchinson62] RSAPv3 uses the Cooper data. The data was re-analyzed to attempt to resolve some long-standing problems with the data in NCHRP22-27. [Ray12]

The results of the re-analysis included generating baseline encroachment frequencies for two-lane undivided, four-lane and multi-lane divided highways. The base conditions for the encroachment frequencies are:

- Posted speed limit of 65 mph,
- Flat ground,
- Relatively straight segment,
- Lane width greater than or equal to twelve feet, and
- Zero major access points per mile.

Deviating from these base conditions requires the use of adjustment factors to calibrate the encroachment frequency to the specific site conditions. The values shown in Table 36 are the base encroachment frequencies for the base conditions.

Table 36. Total Encroachment Frequency by AADT and Highway Type.

AADT (bi-directional)	**2 Lane Undivided** (encr/mi/yr)	**4 Lane Divided** (encr/mi/yr)	**One Way** (encr/mi/yr)
1,000	1.2244	0.8473	0.4236
5,000	2.6514	3.5915	1.7958
10,000	1.8631	5.8435	2.9217
15,000	0.9819	7.1306	3.5653
20,000	1.3091	7.7344	3.8672
25,000	1.6364	7.8650	3.9325
30,000	1.9637	7.6779	3.8389
35,000	2.2909	7.2870	3.6435
40,000	2.6182	6.7749	3.3874
45,000	2.9455	7.6206	3.8103
50,000	3.2728	8.4673	4.2337
55,000	3.6000	9.3140	4.6570
60,000	3.9273	10.1608	5.0804
65,000	4.2546	11.0075	5.5038
70,000	4.5819	11.8542	5.9271
75,000	4.9091	12.7010	6.3505
80,000	5.2364	13.5477	6.7738
85,000	5.5637	14.3944	7.1972
90,000	5.8910	15.2412	7.6206
95,000	6.2182	16.0879	8.0439
100,000	6.5455	16.9346	8.4673

Encroachment models for roads over capacity

Encroachment modeling programs like RSAPv3, RSAP and BCAP make the assumption that traffic is in a free-flow condition. In light of this assumption, showing AADT values within the selection guidelines which exceed the AADT and percent truck values where free-flow is possible would be a misrepresentation. The user of the selection tables should be aware of this assumption when using these tables.

Low-Volume Encroachments

The 1989 GSBR was developed using the program BCAP which NCHRP 22-08 modified into the similar program ABC. Both used a constant encroachment rate based on the Hutchison-Kennedy data. RSAP and RSAPv3, in contrast, use a variable encroachment rate based on the Cooper data. One of the consequences is that the Cooper data has a pronounced "hump" at about 25,000 vehicles/day for divided highways and a pronounced trough at 40,000 vehicles/day. After 40,000 vehicles/day the expected number of encroachments increases monotonically. RSAPv3 generally calculates the mid-life number of encroachments and then uses that value in calculating the expected crash costs. If the mid-life ADT turns out to be on the top of the "hump" the encroachments would be overestimated for the entire life and if the mid-life ADT occurs at the bottom of the "trough" the encroachments would be underestimated. In order to avoid this problem, which only happens at low AADTs, the selection guidelines have been developed using a procedure which calculates the number of encroachments at 10 equally spaced times over the life and then takes the average of these values to estimate the encroachments at the mid-life. This is a more realistic estimate of the average encroachment rate over the life of the project, however, a traffic growth rate must be assumed for the development of these guidelines.

Annual Traffic Growth

Many traffic engineering sources like the Highway Capacity Manual (HCM) use or recommend a default traffic growth rate of two percent. Similarly, the 1989 AASHTO GSBR assumed a two percent traffic growth as well. The selection guidelines have been developed by calculating the expected number of encroachments at 10 points during the life with an assumed two percent growth rate and then averaged to find the mid-life number of encroachments. A procedure to change these assumptions has been provided within the selection process.

Traffic Mix Considerations

The GSBR used the FHWA 13-vehicle classification system in developing its guidelines. According to Harwood *et al*., 5-axle tractor-trailer trucks (i.e., Class 9) alone account for 46.1 percent of the trucks on the nation's highway as measured by the vehicle miles traveled. [Harwood03] Two-axle single-unit trucks (i.e., Class 5) account for 29.5 percent. These two types of trucks alone, then, account for more than 75 percent of the vehicle miles travelled by trucks in the U.S.. After Class 5 and 9, the next highest class is three-axle single-unit trucks (i.e., Class 6) at 5.3 percent and all other classes (i.e., Classes 7, 8, 10-13) each account for less than two percent of the vehicle miles travelled and in many cases less than one percent. In fact, some of the higher classes of trucks (e.g., multi-trailer trucks -- Classes 11 through 13) are either not allowed at all, are allowed only by special permit or restricted to particular routes in many states.

Clearly the single-unit truck and tractor-trailer truck used in Report 350 and MASH are good representative vehicles for trucks since they account for the majority of vehicle miles travelled. All classes of single-unit trucks comprised about 40 percent of the truck vehicle miles travelled and 60 percent were accounted for by a variety of tractor trailer trucks. Generally speaking, there are on average about 1.5 times as many tractor trailer trucks in the average traffic stream as single-unit trucks.

The rollover and penetration algorithm used in RSAPv3 was found to be relatively sensitive to the properties of the heavy vehicles (i.e., weight, dimensions and c.g. location). An unloaded Class 9 tractor trailer truck, for example, weighs about 27,000 lbs whereas a fully loaded Class 9 truck weighs 80,000 lbs and in some States (e.g., Maine) may even weigh more than 100,000 lbs. In addition to selecting the most representative trucks, it is also necessary to select truck properties that accurately reflect the mix of loading conditions experienced on the nation's roadways. Figure 40 shows a cumulative axle-weight distribution of the truck classes from the new Mechanistic-Empirical Pavement Design Guide (MEPDG). [MEPDG04] This data was used to develop the vehicle weights shown in Table 37. The 22,000-lbs single-unit truck used in MASH appears to be a good representation of the 85th percentile two-axle single unit truck whereas the 80,000-lbs tractor trailer is more representative of the 95th percentile for Class 9 truck weights based on the MEPDG. [MEPDG04]

An understanding of the vehicle properties of each vehicle class is needed for the analyses, however, obtaining vehicle properties for vehicles which have not been crash tested would require estimation. Also, the methods used in RSAPv3 to estimate the probability of penetration and rollover are unlikely to work well with vehicles with multiple articulations. Rather than include all 13 vehicle classes in developing the recommendations it seemed more reasonable to focus on the vehicle types with the highest proportion of vehicle-miles travelled. Classes 11 through 13 represent a very small percentage of truck VMT and operate only on selected roadways so there is little harm in ignoring them for general-purpose guidelines.

Table 37. Percetile of Gross Truck Weights for Classes 5 and 9. After [MEPDG04]

Percentile	Gross Weight (lbs)	
	Class 5 Single Unit Trucks	Class 9 Tractor Trailer Trucks
15th	6,754	35,350
50th	11,942	48,822
85th	21,885	62,069

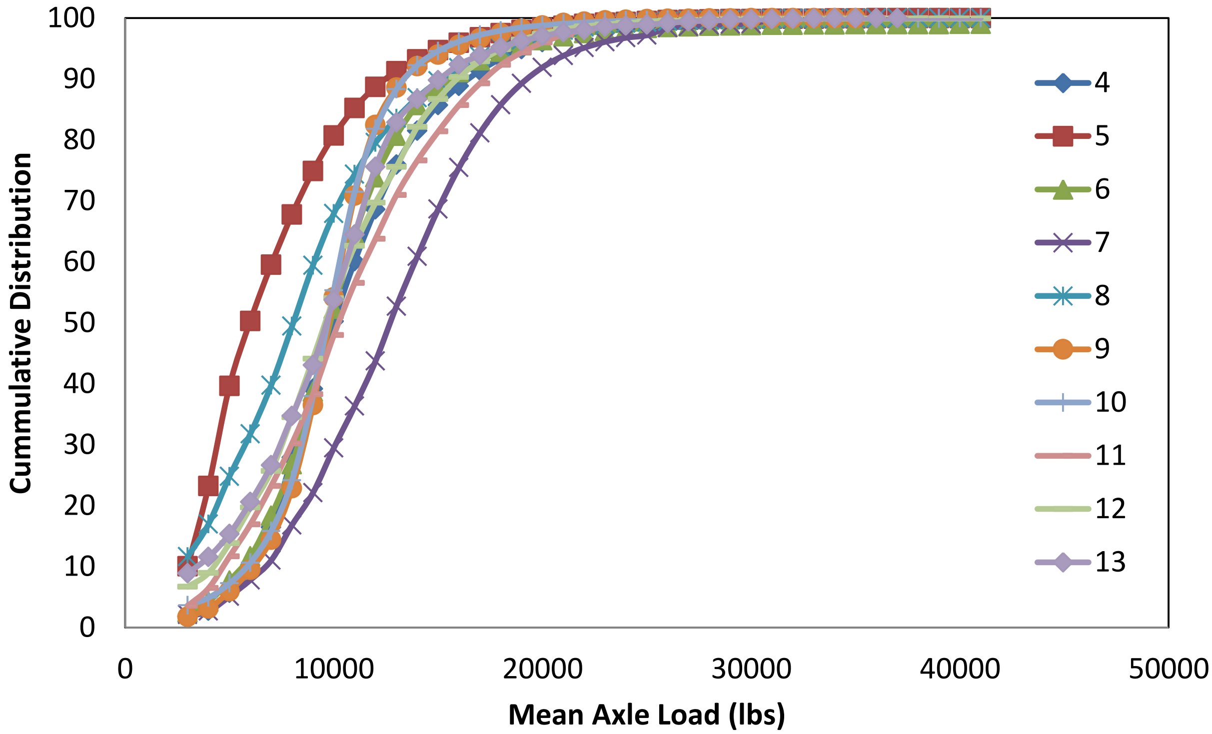


Figure 40. Mean Axle Load by Vehicle Classification.

The capacity and rollover algorithms in RSAPv3 are more sensitive to the c.g. height and weight of the vehicle than the particular class so it appears to be more important to represent the range of weights and c.g. locations for the most common Classes than to represent the average condition of all 13 Classes. The following eight vehicle types were used in developing the selection guidelines:

1. Passenger car,
2. Pickup truck,
3. Light Class 5 single-unit truck (i.e., 15th percentile),
4. Average Class 5 single-unit truck (50th percentile),
5. Heavy Class 5 single-unit truck (85th percentile),
6. Light Class 9 combination tractor-trailer truck (15th percentile),
7. Average Class 9 combination tractor-trailer truck (50th percentile) and
8. Heavy Class 9 combination tractor-trailer truck (95th percentile).

Vehicles 1, 2, 5 and 8 were specifically chosen because they are essentially MASH test vehicles and, therefore, maintain a link to crash testing specifications. The percent of trucks was varied between zero and 40 percent as was done in the 1989 GSBR. Within the defined percent of trucks, the split was 67 percent tractor-trailer trucks and 33 percent single unit trucks. Within the defined percentage of passenger vehicles, the split used was 75 percent passenger cars and 25 percent pickup trucks. This strategy results in the traffic mix for each percentage of trucks shown in Table 38.

Table 38. Vehicle Mix Used in RSAP to Develop the Guidelines.

Percent Trucks	Passenger Cars	Pickup Trucks	Light Tractor Trailer	Average Tractor Trailer	Heavy Tractor Trailer	Light Single Unit	Average Single Unit	Heavy Single Unit
0	75.00	25.00	0.00	0.00	0.00	0.00	0.00	0.00
5	72.50	22.50	1.00	2.00	0.35	0.50	0.65	0.50
10	70.00	20.00	2.00	4.00	0.70	1.00	1.30	1.00
15	67.50	17.50	3.00	6.00	1.05	1.50	1.95	1.50
20	65.00	15.00	4.00	8.00	1.40	2.00	2.60	2.00
25	62.50	12.50	5.00	10.00	1.75	2.50	3.25	2.50
30	60.00	10.00	6.00	12.00	2.10	3.00	3.90	3.00
35	57.50	7.50	7.00	14.00	2.45	3.50	4.55	3.50
40	55.00	5.00	8.00	16.00	2.80	4.00	5.20	4.00

In addition to the mix and weight properties discussed above, the inertial and geometric properties used for the various vehicle classes are also important input parameters. While the vehicle weight distributions for each class are relatively easy to estimate from weigh-in-motion data, other important data like the location of the center of gravity are very difficult to determine for the in-service fleet. The same truck loaded will have very different properties when it is unloaded so the properties chosen for each type of vehicle need to reflect the appropriate proportion of vehicle miles traveled.

The vehicle properties used for the vehicle mix recommended in Table 38 are shown in Table 39. The c.g. height recommended for the tractor-trailer corresponds to the effective c.g. of the trailer, since the trailer rolling over the barrier is generally what pulls the tractor over as well. The effective c.g. is somewhere between the overall c.g. of the vehicle and the c.g. of the trailer. For tractor trailer trucks, the “weight” listed in Table 39 refers to the weight of the King Pin axle rather than the whole vehicle weight. Tractor trailer trucks are articulated and crash test data indicates that the second impact with the King Pin axle is generally the most demanding. A King Pin axle weight of 22,000 lbs corresponds to an 80,000 lbs tractor-trailer truck whereas a 6,800-lbs King Pin axle weight corresponds to an empty tractor trailer truck. The 4.2-ft center of gravity height is based on the crash-test measured value of 80,000-lbs tractor trailer trucks and the 3.4-ft height for the empty tractor trailer truck is based on the empty-weight location of the center of gravity based on a finite element model. The heavy tractor trailer in Table 39 corresponds closely to the MASH 36000V vehicle. The upper bound value (trailer c.g.), which resulted in slightly more rollover barrier incidents (i.e., this method is conservative) was used in the analysis. The overall c.g. and the "effective" c.g. used in the generation of the guidelines are listed below.

Table 39. Vehicle Properties Used in RSAPv3.

RSAPv3 VEHICLES	FHWA Vehicle CLASS	WEIGHT	LENGTH	WIDTH	C.G. Long.	C.G. Hgt
		Lbs	ft	ft	ft	ft
Passenger Vehicles	2	3,300	15.00	5.40	6.00	2.00
Pickup Truck	3	5,000	19.75	6.50	8.50	2.30
Light Tractor Trailer	8-9	16,000	48.00	8.50	12.00	4.80
Average Tractor Trailer	8-13	22,250	48.00	8.50	20.00	4.80
Heavy Tractor Trailer	8-13	37,500	48.00	8.50	20.00	6.00
Light Single Unit Truck	5	6,800	35.00	7.77	12.50	3.40
Average Single Unit Truck	6	12,000	35.00	7.77	12.50	3.40
Heavy Single Unit Truck	7	22,000	35.00	7.77	12.50	4.20

The weights shown for the single-unit trucks in Table 39 correspond to the total weight of the truck. The weight and c.g. height for the heavy single unit truck corresponds to the MASH 10,000S vehicle. The light single unit truck was based on properties of the same vehicle but in an unloaded condition and average load condition, respectively. The light and average SUT values were determined using a finite element model.

Truck Trajectories

RSAPv3, like RSAP and BCAP, bases trajectories on information collected from passenger vehicle encroachments. In the case of BCAP and RSAP, the distributions of encroachment speeds and angles were used to create straight-line trajectories. RSAPv3 takes a more realistic approach where actual vehicle trajectories measured in NCHRP 17-22 were used.[Mak10, Ray12] This allows for a much richer representation of the vehicle trajectory since driver reactions and side-slope conditions are implicitly included in the trajectories.

Assuming that trucks and passenger vehicle share the same encroachment characteristics does not seem reasonable for three reasons:

1. The handling and acceleration/deceleration properties of trucks are very different than passenger cars.
2. Trucks may leave the roadway with different speeds and angles than passenger vehicles.
3. Trucks may encroach at a different rate than passenger vehicles.

Heavy Vehicle Encroachment Angle

Unfortunately, there is no database of heavy vehicle trajectories which includes information relative to departure angle and speed, encroachment rates or trajectories. Using passenger vehicle trajectories would include some trajectories that are clearly difficult to attain for heavy vehicles. Passenger vehicles are smaller, have better braking and acceleration characteristics as well as different inertial properties. BCAP recognized this fact and used the following simple point-mass procedure for limiting the possible encroachment angles based on the vehicle type, offset from the road, available friction and encroachment speed: [AASHTO89]

$$\theta_{max} = \cos^{-1}[1 - \frac{S_o f_{max} g}{V^2}]$$

where Θ_{max} = The maximum likely encroachment angle in degrees,
S_o = The vehicle offset from the edge of the travelled way in feet,
f_{max} = The maximum available coefficient of friction,
g = The gravity constant (i.e., 32.2 ft/s^2) and
V = The encroachment velocity in ft/s.

Table B2 in the AASHTO GSBR presents available friction coefficients of 0.60 for most single-unit trucks and 0.45 for most tractor trailer trucks. [Mak93] The MEPDG indicates that on two-lane in one direction cross-sections 90 percent of trucks travel in the right most lane so the most common offset value in the above equation would be six feet (i.e., half the typical lane width).[MEPDG04]

The NCHRP 17-22 data contains 787 trajectories that were included in the RSAPv3 trajectory tables. The maximum achievable encroachment angle using the equation above was compared to the actual encroachment angle for each of these 787 trajectories. If the actual encroachment angle was greater than the maximum achievable encroachment angle for single-unit trucks or tractor trailer trucks it was excluded from the heavy vehicle analysis. For single-unit trucks, 315 trajectories were found where the actual encroachment angle was less than the maximum achievable and 253 trajectories were found for tractor trailer trucks. The maximum encroachment angle in the trajectory databases used for both single-unit trucks and tractor-trailer trucks is 32 degrees. Coincidentally, BCAP limited all encroachment angles to 36 degrees. The excluded trajectories represent high-angle, high-speed passenger vehicle encroachment trajectories that would be highly unlikely for trucks.

Having different trajectory tables for each type of vehicles is easily accomplished in RSAPv3. Passenger vehicle trajectories (i.e., passenger cars and pickup trucks) are taken from the TrajectoryGrid2 worksheet whereas single-unit truck and tractor-trailer truck trajectories are taken from TrajectoryGrid3 and TrajectoryGrid4, respectively. TrajectoryGrid3 and TrajectoryGrid4 are limited to those trajectories that satisfy the side-friction criteria discussed in the previous paragraphs.

Heavy Vehicle Encroachment Rate

In addition to considering heavy vehicle trajectory differences, the encroachment frequency differences were also examined. RSAPv3 uses the so-called Cooper data to model the vehicle encroachment frequency. The vehicle type is unknown in this dataset since the data was based on tire marks, however, data collectors were instructed to focus on passenger vehicles.

There has been a long-held assumption that heavy vehicles leave the roadway at the same rate as all vehicles. There is no known database of heavy vehicle encroachments and trajectories, however, some organizations have collected heavy vehicle paths and heavy vehicle crash statistics.[NHTSA13;FMCSA12] This research examined the assumption that heavy vehicles encroach onto the roadside at the same rate as passenger vehicles by analyzing a national sample of run-off-road crashes and a detailed regional sample. The results of each are comparable and challenge the assumption that trucks encroach at the same rate.

The national crash data examined for this analysis includes: the Federal Motor Carrier Safety Administration (FMCSA) national dataset of commercial truck crashes from 2002 through 2011; the U.S. Department of Transportation National Highway Safety Administration (NHTSA) annual *Traffic Safety Facts reports* for the years 2002 through 2010; and the Federal Highway Administration (FHWA) Highway Statistics website for 2002 through 2010.[NHTSA12; FMCSA12; FHWA12c] The detailed regional dataset used for this analysis includes crash and traffic records for 100 miles of the New Jersey Turnpike from 2005 through 2008.[Plan4S11]

NATIONAL DATA

The NHTSA Traffic Safety Facts is a nationwide database of different crash types, including run-off road crashes. The data is presented by crash location in relation to the roadway. The available fields are: "On Roadway," "Off Roadway," "Shoulder," "Median," "Other/Unknown," and "Total". The fields "Off Roadway," "Shoulder," and "Median" were used to represent run-off road crashes, while the "Total" field was used to represent all crashes.[NHTSA13]

The FMCSA database contained only truck and bus crashes. Event IDs that were labeled as "Non collision ran off road", "Non collision overturn (rollover)", "Non collision cross median/centerline", or "Collision involving fixed object" were considered to be run-off road crashes. FMCSA data for the years 2002-2011 was used.[FMCSA12]

Traffic counts by vehicle classifications were obtained from the FHWA Highway Statistics Series. "Single-Unit 2-Axle 6-Tire or More and Combination Trucks" were used for trucks, "Buses" was used for buses, and "All Motor Vehicles" was used with the NHTSA Traffic Safety Facts data. Beginning in 2007 states were required to report motorcycle data and FHWA implemented a new methodology to calculate traffic counts, therefore, some of the VMT values given for the different years may not fit into the trend lines that adjacent years fit into. This is apparent in Table 40 and should be kept in mind when interpreting the results.[FHWA12c]

REGIONAL DATA

The New Jersey Turnpike has a continuous TL5 concrete median barrier from milepost 1.2 to 104.7. Since the median barrier is continuous and relatively close to the left shoulder it is similar to a direct measure of primary and opposing left encroachments. Crash data for left exiting vehicles that were reported as striking the "Concrete Traffic Barrier" under any of the four possible sequence of events fields were considered in the analysis. The frequency of these events was determined for all crashes on this section of highway and for heavy vehicle crashes. Heavy vehicles were defined as one of the following: "Bus/Large Van (9 or more seats)", "Recreation Vehicle", "Single Unit (2 axle)", "Single Unit (3+ axle)", "Single Unit Truck w/Trailer", "Tractor Double", "Tractor Semi-Trailer", "Truck Tractor (Bobtail)", "Tractor Triple", and "Other Truck". Crashes were then separated by year and the milepost where the crash occurred. The highway segments were defined by the known traffic volumes for given mileposts, not geometrics. The hundred million vehicle miles traveled (100 MVMT) was calculated for each segment and year, using the following equation:

$$100\ MVMT = \frac{(AADT)(365)(segment\ length\ in\ miles)(\#years)}{100{,}000{,}000}$$

where: AADT = Annual Average Daily Traffic, and
100 MVMT = Hundred Million Vehicle Miles Traveled.

After the vehicle-miles travelled was calculated for each segment, the crash rate was calculated for each segment and year for both heavy vehicles and all vehicles as follows:

$$CR = \frac{\#crashes}{(100\ MVMT)}$$

where: CR = Crash Rate.

RESULTS

The vehicle miles traveled data collected by the FHWA are shown in the second through fourth columns of Table 40. Table 40 also contains the number of crashes and the crash rates for truck, bus, and truck/bus for the years 2002 through 2010. These national values represent all possible crashes from vehicles which may have encroached from all possible directions (i.e., primary right, opposing right, primary left, opposing left) while the NJTA data only include left encroachments.

A dramatic decrease in crash rates can be seen from the years 2006 – 2007, presumably caused at least in part by the methodological changes for reporting MVMT data implemented in 2007.

Table 40. National Traffic Volumes, Crashes, and Crash Rates by Year.

Year	Trucks 100 MVMT	Bus 100 MVMT	Truck & Bus 100 MVMT	Truck and Bus ROR Crashes		Bus Only ROR Crashes		Truck Only ROR Crashes	
				# ROR	Crash/ 100 MVMT	# Bus ROR	Crash/ 100 MVMT	# Truck ROR	Crash/ 100 MVMT
2002	2,146	68.45	2,214	24,483	11.06	683	9.98	23,800	11.09
2003	2,179	67.82	2,247	27,102	12.06	772	11.38	26,330	12.08
2004	2,208	68.01	2,276	32,234	14.16	902	13.26	31,332	14.19
2005	2,225	69.80	2,295	33,000	14.38	964	13.81	32,036	14.40
2006	2,225	67.83	2,293	31,754	13.85	946	13.95	30,808	13.85
2007	3,042	145.16	3,187	33,051	10.37	1,025	7.06	32,026	10.53
2008	3,107	148.23	3,255	36,954	11.35	1,471	9.92	35,483	11.42
2009	2,880	143.58	3,024	29,658	9.81	1,295	9.02	28,363	9.85
2010	2,866	137.89	3,004	28,026	9.33	997	7.23	27,029	9.43
Avg	**2,542**	**101.86**	**2,644**	**30,696**	**11.61**	**1,006**	**9.88**	**29,690**	**11.68**

Table 41 was developed from NHTSA FARS/GES data as a representation of run-off-road crash rates for all vehicles.

Table 41. NHTSA FARS/GES Crash Data and Crash Rates.

Year	**100 MVMT**	**Total Crashes**	**"All Crash" Crash Rate (per 100 MVMT)**	**Crashes "Off Road"**	**Crashes "Shldr"**	**Crashes "Med"**	**Total ROR Crashes**	**ROR Crash Rate (crashes per 100 MVMT)**
2002	28,555	6,316,000	221.19	790,000	25,000	122,000	1,116,000	39.08
2003	28,902	6,328,000	218.95	865,000	34,000	127,000	1,143,000	39.55
2004	29,648	6,181,000	208.48	969,000	34,000	156,000	1,128,000	38.05
2005	29,894	6,159,000	206.03	913,000	59,000	154,000	1,151,000	38.50
2006	30,144	5,973,000	198.15	859,000	65,000	143,000	1,067,000	35.40
2007	30,311	6,024,000	198.74	948,000	53,000	150,000	1,126,000	37.15
2008	29,765	5,811,000	195.23	919,000	46,000	163,000	1,159,000	38.94
2009	29,535	5,505,000	186.39	954,000	32,000	157,000	1,026,000	34.74
2010	29,665	5,419,000	182.67	946,000	33,000	137,000	937,000	31.59
Avg	**29,602**	**5,968,444**	**201.76**	**907,000**	**42,333**	**145,444**	**1,094,778**	**37.00**

Table 42 contains a summary of the ROR crash rates obtained from both the FMCSA (i.e., heavy vehicle ROR crashes) and NHTSA FARS/GES data sets (i.e, all vehicle ROR crashes). The last row contains a simple comparison of the crash rates defined as:

$$\frac{CR_T}{CR_A} = TCRM$$

where: CR_T = Crash Rate of Trucks and Buses (crashes / 100 MVMT),
CR_A = Crash Rate of All Vehicles (crashes / 100 MVMT), and
TCRM = Truck/Bus Crash Rate Multiplier (dimensionless).

Table 42. National Heavy Vehicle Crashes Per 100MVMT.

Year	ROR Bus	ROR Truck	ROR Truck and Bus	ROR All Vehicles	TCRM (CR_{TB}/CR_A)
2002	9.98	11.09	11.06	39.08	0.28
2003	11.38	12.08	12.06	39.55	0.30
2004	13.26	14.19	14.16	38.05	0.37
2005	13.81	14.40	14.38	38.50	0.37
2006	13.95	13.85	13.85	35.40	0.39
2007	7.06	10.53	10.37	37.15	0.28
2008	9.92	11.42	11.35	38.94	0.29
2009	9.02	9.85	9.81	34.74	0.28
2010	7.23	9.43	9.33	31.59	0.29
Avg	**9.88**	**11.68**	**11.61**	**37.00**	**0.31**

This analysis suggests that the long-held assumption that heavy vehicles encroach onto the roadside at the same rate as all vehicles is false. In fact, heavy vehicles appear to encroach at an average rate of approximately one-third of all vehicles.

REGIONAL ANALYSIS

A review of a regional sample of data was conducted to validate the findings from the National average statistics. The regional sample chosen had detailed traffic volumes, roadway inventory data, and only one hazard was considered (i.e., concrete median barrier). The results for the New Jersey Turnpike analysis are shown in Table 43, Table 44, Table 45, and Table 46 for the years 2005, 2006, 2007, and 2008, respectively. Only primary left (PL) and opposing left (OL) encroachment directions were considered and the segments are defined solely by available traffic volumes.

The truck/bus crash rates of the New Jersey turnpike appear to be approximately 28 percent of the crash rates for all vehicles which is similar to the 0.31 from the nationwide analysis. This analysis supports the national analysis finding that heavy vehicles have a lower encroachment rate than “all vehicles.”

Table 43. NJ Turnpike Ran Off Road Left Crash Rates, 2005.

Link and Posted Speed Limit	**Mileposts**	**Miles**	**Total Crash #**	**Truck Crash #**	**ADT**	**%T**	**100 MVMT**	**CR_A (All)**	**CR_T (Trk/Bus)**
1 - 2 (65)	1.2 - 12.9	11.7	26	0	45830	15.3	1.96	13.28	0.00
2 - 3 (65)	12.9 - 26.1	13.2	51	2	49177	15.5	2.37	21.52	5.45
3 - 4 (65)	26.1 - 34.5	8.4	31	2	58486	15.3	1.79	17.29	7.29
4 - 5 (65)	34.5 - 44	9.5	45	2	73616	15.1	2.55	17.63	5.17
5 - JCT (65)	44 - 51	7	33	2	79833	14.9	2.04	16.18	6.60
JCT - 7 (65)	51 - 53.3	2.3	20	0	109671	15.5	0.92	21.72	0.00
7 - 7A (65)	53.3 - 60	6.7	62	3	120474	16.2	2.95	21.04	6.29
7A - 8 (65)	60 - 67.6	7.6	57	2	132809	16.7	3.68	15.47	3.25
8 - 8A (65)	67.6 - 73.7	6.1	62	4	137157	16.2	3.05	20.30	8.08
8A - 9 (65)	73.7 - 83.3	9.6	61	6	160390	15.4	5.62	10.85	6.95
9 - 10 (65)	83.3 - 88.1	4.8	31	1	202341	13.6	3.55	8.74	2.07
10 - 11 (65)	88.1 - 90.6	2.5	18	0	187670	13.7	1.71	10.51	0.00
11 - 12 (65)	90.6 - 95.9	5.3	38	0	224591	14.0	4.34	8.75	0.00
12 - 13 (65)	95.9 - 97.3	1.4	7	0	235830	14.4	1.21	5.81	0.00
12 - 13 (55)	97.3 - 99.9	2.6	39	0	235830	14.4	2.24	17.43	0.00
13 - 13A (55)	99.9 - 101.6	1.7	31	0	250812	14.8	1.56	19.92	0.00
13A - 14 (55)	101.6 - 104.7	3.1	33	1	223337	15.3	2.53	13.06	2.59
Average:			**26**	**1.47**	**148697**	**15.1**	**2.59**	**15.27**	**3.16**

Table 44. NJ Turnpike Ran Off Road Left Crash Rates, 2006.

Link and Posted Speed Limit	**Mileposts**	**Miles**	**Total Crash #**	**Truck Crash #**	**ADT**	**% T**	**100 MVMT**	**CR_A (All)**	**CR_T (Tks/Bus)**
1 - 2 (65)	1.2 - 12.9	11.7	35	0	46789	15.7	2.00	17.52	0.00
2 - 3 (65)	12.9 - 26.1	13.2	40	1	50085	15.8	2.41	16.58	2.62
3 - 4 (65)	26.1 - 34.5	8.4	34	1	59345	15.5	1.82	18.69	3.55
4 - 5 (65)	34.5 - 44	9.5	56	0	74484	15.4	2.58	21.68	0.00
5 - JCT (65)	44 - 51	7	20	0	80861	15.2	2.07	9.68	0.00
JCT - 7 (65)	51 - 53.3	2.3	15	0	110908	15.7	0.93	16.11	0.00
7 - 7A (65)	53.3 - 60	6.7	49	2	121384	16.3	2.97	16.51	4.14
7A - 8 (65)	60 - 67.6	7.6	68	1	133548	16.8	3.70	18.36	1.60
8 - 8A (65)	67.6 - 73.7	6.1	60	1	137794	16.3	3.07	19.56	2.00
8A - 9 (65)	73.7 - 83.3	9.6	51	1	160763	15.5	5.63	9.05	1.14
9 - 10 (65)	83.3 - 88.1	4.8	49	3	203277	13.7	3.56	13.76	6.13
10 - 11 (65)	88.1 - 90.6	2.5	26	1	189709	13.8	1.73	15.02	4.19
11 - 12 (65)	90.6 - 95.9	5.3	50	1	228790	14.0	4.43	11.30	1.61
12 - 13 (65)	95.9 - 97.3	1.4	7	0	240350	14.4	1.23	5.70	0.00
12 - 13 (55)	97.3 - 99.9	2.6	47	5	240350	14.4	2.28	20.61	15.23
13 - 13A (55)	99.9 - 101.6	1.7	43	5	255416	14.8	1.58	27.13	21.28
13A - 14 (55)	101.6 - 104.7	3.1	43	3	226013	15.4	2.56	16.81	7.62
Average:			**41**	**1.47**	**150580**	**15.2**	**2.62**	**16.12**	**4.18**

Table 45. NJ Turnpike Ran Off Road Left Crash Rates, 2007.

Link and Posted Speed Limit	**Mileposts**	**Miles**	**Total Crash #**	**Truck Crash #**	**ADT**	**%T**	**100 MVMT**	**CR_A (All)**	**CR_T (Trk/Bus)**
1 - 2 (65)	1.2 - 12.9	11.7	24	1	47325	16.0	2.02	11.88	3.08
2 - 3 (65)	12.9 - 26.1	13.2	40	1	50906	16.2	2.45	16.31	2.52
3 - 4 (65)	26.1 - 34.5	8.4	38	3	60384	15.9	1.85	20.53	10.21
4 - 5 (65)	34.5 - 44	9.5	55	1	75790	15.7	2.63	20.93	2.42
5 - JCT (65)	44 - 51	7	38	1	81964	15.5	2.09	18.15	3.08
JCT - 7 (65)	51 - 53.3	2.3	16	0	112538	16.1	0.94	16.94	0.00
7 - 7A (65)	53.3 - 60	6.7	49	4	122467	16.6	2.99	16.36	8.05
7A - 8 (65)	60 - 67.6	7.6	68	3	134428	17.1	3.73	18.24	4.69
8 - 8A (65)	67.6 - 73.7	6.1	38	5	138625	16.7	3.09	12.31	9.73
8A - 9 (65)	73.7 - 83.3	9.6	90	2	161555	15.8	5.66	15.90	2.23
9 - 10 (65)	83.3 - 88.1	4.8	45	0	204774	13.9	3.59	12.54	0.00
10 - 11 (65)	88.1 - 90.6	2.5	30	0	193081	14.0	1.76	17.03	0.00
11 - 12 (65)	90.6 - 95.9	5.3	41	0	232567	14.2	4.50	9.11	0.00
12 - 13 (65)	95.9 - 97.3	1.4	10	0	244033	14.4	1.25	8.02	0.00
12 - 13 (55)	97.3 - 99.9	2.6	53	3	244033	14.4	2.32	22.89	8.97
13 - 13A (55)	99.9 - 101.6	1.7	39	4	258419	14.9	1.60	24.32	16.69
13A - 14 (55)	101.6 - 104.7	3.1	39	2	227390	15.5	2.57	15.16	5.01
Average:			**42**	**1.76**	**152369**	**15.5**	**2.65**	**16.27**	**4.51**

Table 46. NJ Turnpike Ran Off Road Left Crash Rates, 2008.

Link and Posted Speed Limit	Mileposts	Miles	Total Crash #	Truck Crash #	ADT	%T	100 MVMT	CR_A (All)	CR_T (Trk/ Bus)
1 - 2 (65)	1.2 - 12.9	11.7	21	0	45640	15.4	1.95	10.77	0.00
2 - 3 (65)	12.9 - 26.1	13.2	43	1	48889	15.5	2.36	18.26	2.73
3 - 4 (65)	26.1 - 34.5	8.4	36	0	58134	15.4	1.78	20.20	0.00
4 - 5 (65)	34.5 - 44	9.5	39	1	73248	15.3	2.54	15.36	2.57
5 - JCT (65)	44 - 51	7	31	3	78947	15.1	2.02	15.37	9.85
JCT - 7 (65)	51 - 53.3	2.3	16	1	109200	15.7	0.92	17.45	6.95
7 - 7A (65)	53.3 - 60	6.7	48	1	118768	16.2	2.90	16.53	2.12
7A - 8 (65)	60 - 67.6	7.6	56	2	130011	16.8	3.61	15.53	3.30
8 - 8A (65)	67.6 - 73.7	6.1	46	1	134028	16.3	2.98	15.41	2.05
8A - 9 (65)	73.7 - 83.3	9.6	49	2	155581	15.5	5.45	8.99	2.36
9 - 10 (65)	83.3 - 88.1	4.8	40	1	197707	13.6	3.46	11.55	2.12
10 - 11 (65)	88.1 - 90.6	2.5	10	1	187069	13.9	1.71	5.86	4.23
11 - 12 (65)	90.6 - 95.9	5.3	38	4	225401	14.0	4.36	8.71	6.55
12 - 13 (65)	95.9 - 97.3	1.4	8	0	236947	14.4	1.21	6.61	0.00
12 - 13 (55)	97.3 - 99.9	2.6	62	3	236947	14.4	2.25	27.57	9.27
13 - 13A (55)	99.9 - 101.6	1.7	34	2	251121	15.1	1.56	21.82	8.51
13A - 14 (55)	101.6 - 104.7	3.1	35	2	221602	15.6	2.51	13.96	5.12
Average:			**36**	**1.47**	**147602**	**15.2**	**2.56**	**14.70**	**3.98**

Table 47. Truck/Bus Crash Rate Multipliers, by Year and Link.

Link and Posted Speed Limit	Mileposts	TCRM 2005	TCRM 2006	TCRM 2007	TCRM 2008
1 - 2 (65)	1.2 - 12.9	0.00	0.00	0.26	0.00
2 - 3 (65)	12.9 - 26.1	0.25	0.16	0.15	0.15
3 - 4 (65)	26.1 - 34.5	0.42	0.19	0.50	0.00
4 - 5 (65)	34.5 - 44	0.29	0.00	0.12	0.17
5 - JCT (65)	44 - 51	0.41	0.00	0.17	0.64
JCT - 7 (65)	51 - 53.3	0.00	0.00	0.00	0.40
7 - 7A (65)	53.3 - 60	0.30	0.25	0.49	0.13
7A - 8 (65)	60 - 67.6	0.21	0.09	0.26	0.21
8 - 8A (65)	67.6 - 73.7	0.40	0.10	0.79	0.13
8A - 9 (65)	73.7 - 83.3	0.64	0.13	0.14	0.26
9 - 10 (65)	83.3 - 88.1	0.24	0.45	0.00	0.18
10 - 11 (65)	88.1 - 90.6	0.00	0.28	0.00	0.72
11 - 12 (65)	90.6 - 95.9	0.00	0.14	0.00	0.75
12 - 13 (65)	95.9 - 97.3	0.00	0.00	0.00	0.00
12 - 13 (55)	97.3 - 99.9	0.00	0.74	0.39	0.34
13 - 13A (55)	99.9 - 101.6	0.00	0.78	0.69	0.39
13A - 14 (55)	101.6 - 104.7	0.20	0.45	0.33	0.37
Average:		**0.20**	**0.22**	**0.25**	**0.28**

SUMMARY

The regional analysis found that, on average, heavy vehicles experienced an encroachment rate that was 28 percent of the rate for all vehicle types. The national analysis found heavy vehicle ROR crashes occur at a rate of 31 percent of the all-vehicle run-off-road crashes. When modeling run-off-road crashes, it appears necessary to reduce the number of heavy vehicle encroachments by about 30 percent to account for the smaller likelihood of these vehicles encroaching.

There are several possible reasons for this reduced likelihood of heavy vehicles encroaching. First, heavy vehicles are not as maneuverable as passenger vehicles and they have much more restrictive acceleration and deceleration capabilities. Second, heavy vehicles are operated by trained professional drivers who must operate their vehicles in accordance with specific requirements including hours of rest available to the driver. While the causes of this reduced encroachment rate are speculative, the data examined demonstrates that the number of encroachments that can be expected from heavy vehicles is only about 30 percent of passenger vehicles.

Encroachment Adjustments for Site-Specific Characteristics

The 1989 AASHTO GSBR includes three adjustment factors; a horizontal curvature adjustment factor, a grade adjustment factor and an adjustment factor for deck height and under-structure conditions.

The adjustments for the grade and horizontal curvature from the 1989 AASHTO GSBR are shown in Table 48 and Table 49. These values were also used in RSAP 2.0.3 as well as the latest version of RSAP, RSAPv3. While NCHRP 17-54 is in the process of updating these adjustments, the values shown in Table 48 and Table 49 are the best available data at the present time. These adjustment factors should be used until suitable replacements can be made.

Table 48. Grade (F_{grade}) Adjustment Factor.

Grade	F_{grade}
<= -6	2
-6	2
-4	1.5
-2	1
>= -2	1

Table 49. Horizontal Curve (F_{hcurv}) Adjustment Factor.

Degree of Curvature	Radius of Curvature	F_{hcurv}
<= -6	-9545	4.0
-5	-1145	3.0
-4	-1430	2.0
-3	-1910	1.0
0	∞	1.0
3	1910	1.0
4	1430	1.3
5	1145	1.7
>= 6	955	2.0

RSAPv3 also includes adjustment factors for the number of lanes, lane width, posted speed limit and the access density. These adjustment factors have been incorporated into the selection guidelines.

The 1989 AASHTO GSBR bridge height adjustment to adjust for the severity of the under-bridge conditions was applied in the 1989 AASHTO GSBR inappropriately to the number of expected encroachments since the increase in severity should only apply to those cases where the vehicle penetrates the bridge railing. The GSBR applies the factor to all crashes regardless of whether a penetration occurred or not. In RSAPv3, the condition of the area under the bridge is accounted for by a special edge hazard. The severity of crossing the special edge is only calculated and included for those cases where the vehicle crosses the special edge by penetrating, rolling over or vaulting over the bridge railing. The area under the bridge is accounted for in the selection tables and described in full in a later section. It is not treated as an encroachment adjustment factor.

Bridge Shoulder Offset

The 1989 GSBR and NCHRP 22-08 provided the four offset distances (i.e., bridge shoulders) in the selection guidelines:

- 0-3 ft (nominal 1 ft)
- 3-7 ft (nominal 4 ft)
- 7-12 ft (nominal 8 ft)
- >12 ft (nominal 12 ft)

The effect of shoulder width on crash severity has been one of the more interesting features of this research. BCAP, ABC and the 1989 AASHTO GSBR all assume that encroachments follow a straight path and are in a state of constant deceleration. Under these assumptions, any trajectory has a lower severity (i.e., slower speed and same angle) the farther the vehicle travels so wider shoulder widths always result in reduced crash severity.

RSAPv3, as discussed above, uses actual trajectories collected in NCHRP 17-22. These trajectories are sometimes straight, sometimes curved to the left, sometimes curved to the right and sometimes have compound curvatures. These various trajectory curvatures are due to the driver's response to leaving the roadway. In early RSAPv3 runs it was discovered that the crash severity for a particular trajectory sometimes increased as the shoulder width increased. The reason is that some trajectories leave the road with the angle relative to the roadway increasing as the trajectory travels further off the road. If, for example, the vehicle left the roadway with a speed and angle that were close to indicating rollover or penetration with a one-ft offset, the speed and angle at a four-ft offset sometimes was enough to indicate penetration or rollover. Simply stated, the vehicle sometimes would not penetrate or rollover when the offset was one foot but would penetrate or rollover at a higher offset because the angle was increasing. For example, when performing runs for a one foot shoulder with impacts by the average single unit truck, RSAPv3 selected 40 trajectories that matched the site conditions (i.e., speed limit, flat side slope, horizontal curvature and grade). Of these 40 trajectories, five penetrated or rolled over 24-inch high low-profile barrier. When the offset was increased to four feet, seven trajectories resulted in rollover or penetration. The reason was that the angle for some of the trajectories was increasing so for some trajectories, a rollover/penetration did not occur at one-ft but did occur at four-ft due to the higher angle.

The offset where the equivalent adjusted ADT is minimized appears to be at about 8 ft. As the offset further increases the ADT and percent truck values increase slightly as well so it appears that the 8-ft offset is the limiting value. Since the worst-case value always occurs at a shoulder offset of 8 ft, the selection guidelines have been based on the shoulder offsets at 8 ft and the shoulder offset should not be a consideration in the selection. Doing so is conservative for narrower shoulders. If, for example, an agency built a bridge with a 12-ft shoulder anticipating converting it to an 11-ft lane and 1-ft shoulder in the future, the bridge railing selected would not change since it is based on the more critical 8-ft shoulder offset.

Crash

The probability of a collision given that a vehicle has encroached onto the roadside or median is determined in RSAPv3 by directly projecting reconstructed vehicle trajectories onto the roadside or median and determining if the trajectory intersects the position of any hazard, in this case bridge railings. Figure 41 provides a simplified representation of the steps which the crash prediction module takes to determine if a collision occurs; if a terrain rollover occurs; or if nothing happens and the encroachment results in a non-crash event.[Ray12]

As shown in Figure 41, when examining a trajectory, there are many possible outcomes. For example, a trajectory may interact with a roadside hazard, then either stop in contact, penetrate, or be redirected. The probability of each of these events is calculated and the outcome of each sub-event is evaluated. For example, if the vehicle penetrates the barrier, the trajectory is followed further to determine if it interacts with another hazards or results in a rollover. If the trajectory is redirected, the redirected paths are evaluated.

Predicted Penetration, Rollovers and Vaults

The ability to reasonably predict the number of roll-over-the-barrier, vault the barrier and penetrate-the-barrier crashes is critical to obtaining correct crash costs and so, is also critical to obtaining correct results. In RSAPv3, a penetration implies a complete structural failure of the barrier which allows the vehicle to pass through. A rollover-the-barrier is when the vehicle rolls over the barrier and off the bridge whereas a redirection rollover is one in which the vehicle rolls over but remains on the bridge. The capacity values for TL2 through Report 350 TL 4 used in developing these guidelines are based on taking 1.6 times the recommended AASHTO bridge railing design loads from Table A13.2-1 of the LRFD Bridge Design Specification as shown in Table 50. [AASHTO12] The values used for MASH TL4 and TL5 are based on recommendations from a recent TXDOT study that examined what recommended values should be used for the new MASH test criteria. [AASHTO09, Sheihk11] When compared to available crash tests this tends to provide a good estimate of the lower bound of crash tested strengths when complete barrier failure might occur.

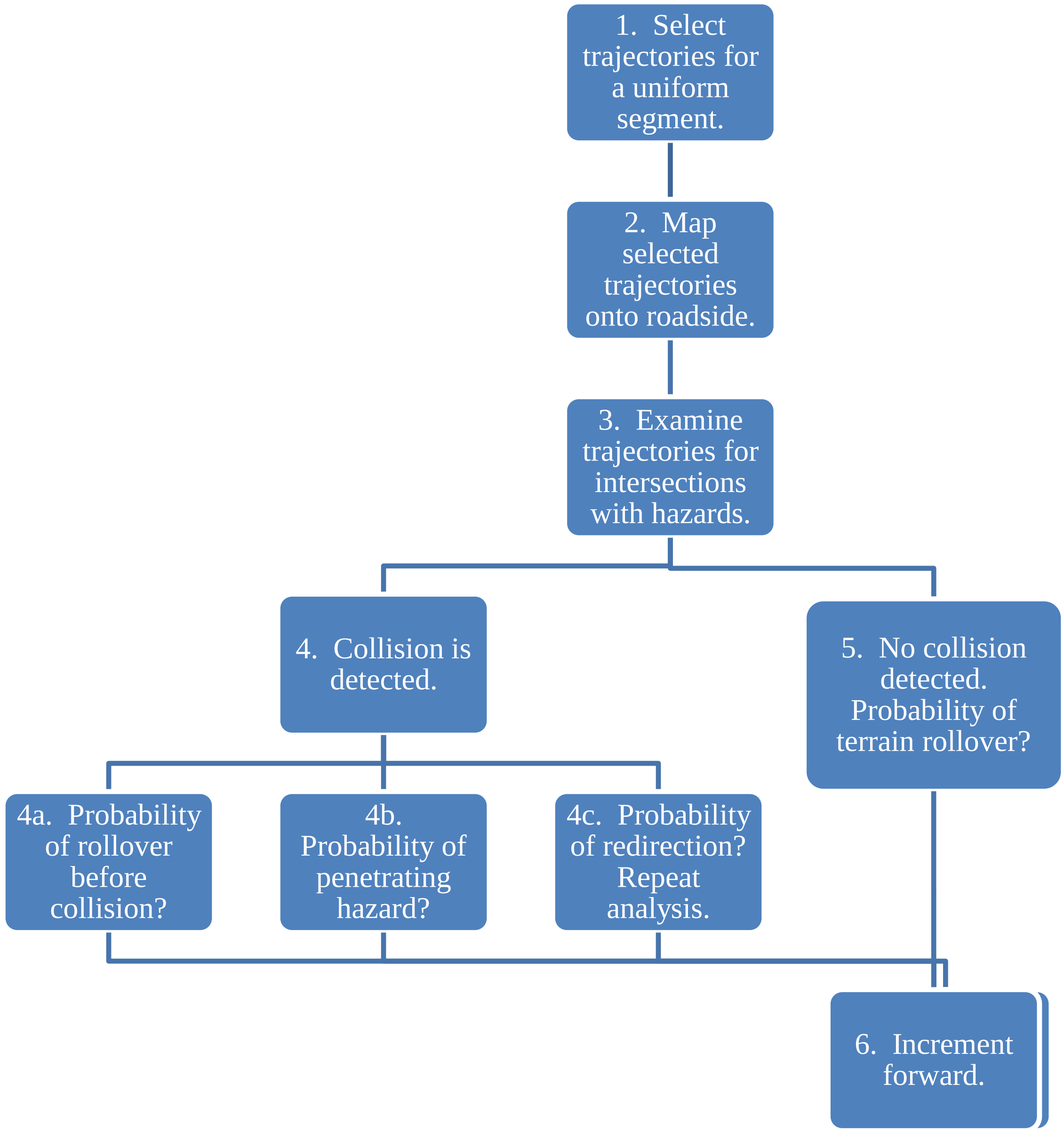


Figure 41. RSAPv3 Crash Prediction Module Flow Chart.

Table 50. Bridge Railing Load Capacities.

Test Level	Barrier Height	AASHTO LRFD	BCAP	ABC	RSAPv3
		Load Capacity			
	(inches)	(kips)	(kips)	(kips)	(kips)
MASH TL 2	24	--	--	--	43.2
MASH TL 3	27	27	15	30	43.2
R350 TL 4	32	54	35	64	86.4
MASH TL 4†	36	80	--	--	128.0
MASH TL 5†	42	160	55	108	256.0

† These values are being considered by but have not been adopted by AASHTO for a future revision of the LRFD Bridge Design Specifications. They are based on [Sheihk11].

NCHRP 22-08 performed a crash study of bridge railing crashes in Texas. [Mak93] As discussed in the literature review, there were numerous problems analyzing and interpreting the data as well as some serious coding issues. In the end, however, it appeared that for bridge railings installed after 1965 the percentage of all vehicle types that penetrated or rolled over the bridge railing was three percent (i.e., 1.1 percent penetrated and 1.9 percent rolled over the bridge railing). Since NCHRP 22-08 was completed in 1993 it is presumed that the majority of bridge railings in the TXDOT data would, at best, represent PL1 bridge railings in the AASHTO GSBR or a mixture of TL2 and TL3 railings in NCHRP Report 350 or MASH.

RSAPv3 models the edge of bridge hazard as a line on the back side of the bridge railing. If a vehicle crosses this line for any reason, the "hazard" is consider contacted. Vehicles can cross this line by penetrating through the bridge railing, vaulting over it or rolling over it. These types of events are called penetration-rollover-vaults (PRV) in RSAPv3. Consider an example case of a 60 mi/hr highway, RSAPv3 results in the penetration, roll-over-the-barrier and redirection rollover values shown in Table 51. Table 51 also compares the RSAPv3 predictions to the average values for passenger vehicles (i.e., passenger cars and pickups), the average single-unit truck and the average tractor trailer truck. Unfortunately, there is no crash data available for the low-profile concrete bridge railing (i.e., TL2) since there is only a small inventory of that particular barrier installed. In general, RSAPv3 tends to slightly over predict roll-over-barrier and penetration collisions so the method is still conservative.

Approximately 6,450 bridge rail crashes in Pennsylvania, Ohio, and Nebraska were reviewed in this study as described earlier. Only eighty-nine of these events (i.e., 1.4 percent) were heavy vehicle crashes. Unfortunately, the crash data does not identify specific types or loadings of vehicles but if the RSAPv3 predictions are weighted by the vehicle mix percentages discussed above they can be combined into a likely penetration and rollover percentage that can be compared to the crash data collected in this project and in NCHRP 22-08 (see Table 52). Given that the TXDOT data was collected in the 1980's it is assumed that bridges constructed after 1965 would likely have bridge railings that conform to the 1989 GSBR PL1 but there was probably still relatively little inventory of PL2 or PL3. It is assumed in Table 51, therefore, that the crash data is mainly representative of PL1 railings.

While the rollover and penetration percentages predicted by RSAPv3 are still higher than observed crash data indicate, they appear to be much more reasonable than the predictions of either BCAP or ABC. Given the acknowledged unreliability of the NCHRP 22-08 crash data and the relatively few heavy vehicle cases that could be obtained in the current study, the RSAPv3 penetration-rollover-vault procedures were used in the development of these guidelines. The results are on the conservative side but appear to be reasonable given the uncertainty in the crash data. One should also consider that the capacity loads represent the low end of the spectrum – specific bridge railings could be much stronger based on the geometric and reinforcement details used.

Table 51. RSAPv3 Predictions of Penetration and Rollovers compared to NCHRP 22-08 TXDOT Crash Data.

Vehicle	Bridge Rail Height (in)	RSAPv3 Prediction				NCHRP 22-12(3) Crash Data			
		Penetration	Rollover Barrier	Redirection Rollover	Redirected	Penetration	Rollover Barrier	Redirection Rollover	Redirected
Passenger Car	24	0.3	0.0	2.0	97.7				
	27	0.3	0.0	2.0	97.7				
	32	0.0	0.0	2.0	98.0				
	36	0.0	0.0	2.0	98.0				
	42	0.0	0.0	2.0	98.0				
Pickup Truck	24	4.3	0.0	2.0	93.7				
	27	4.4	0.0	2.0	93.6				
	32	0.0	0.0	2.0	98.0				
	36	0.0	0.0	2.0	98.0				
	42	0.0	0.0	2.0	98.0				
All Passenger Vehicles	24	1.3	0.0	2.0	96.7	--	--	--	--
	27	1.3	0.0	2.0	96.7	5.0	5.6	18.8	70.6
	32	4.3	0.0	2.0	93.7	0.6	1.9	5.6	95.7
	36	2.5	0.0	2.0	98.0	0.1	0.3	2.7	96.9
	42	0.9	0.0	2.0	98.0	0.0	0.2	3.2	96.6
Light Single Unit Truck	24	3.4	12.5	4.0	80.2				
	27	3.8	2.9	3.5	89.8				
	32	0.0	5.0	0.0	95.0				
	36	0.0	0.0	0.0	100.0				
	42	0.0	0.0	0.0	100.0				
Average Single Unit Truck	24	8.8	7.4	3.8	80.1	--	--	--	--
	27	9.3	0.6	2.4	87.7	12.2	4.9	19.5	63.4
	32	2.1	3.1	0.0	94.8	2.4	4.9	2.4	90.2
	36	0.0	0.0	0.0	100.0	0.0	2.4	4.9	92.7
	42	0.0	0.0	0.0	100.0	0.0	0.0	4.9	95.1
Heavy Single Unit Truck	24	33.5	2.0	15.0	49.5				
	27	34.4	0.0	6.2	59.4				
	32	11.5	7.3	4.4	76.8				
	36	5.5	1.2	3.9	89.5				
	42	0.1	0.2	0.0	99.7				
Light Tractor Trailer Truck	24	22.7	14.3	10.7	52.4				
	27	23.1	5.6	10.6	60.7				
	32	4.5	16.2	6.1	73.2				
	36	2.5	13.7	4.4	79.3				
	42	0.0	3.0	3.3	93.7				
Average Tractor Trailer Truck	24	7.5	15.1	19.1	58.2	--	--	--	--
	27	8.0	9.2	19.0	63.7	12.2	4.9	19.5	63.4
	32	1.1	14.1	11.4	73.4	2.4	4.9	2.4	90.2
	36	0.0	4.9	14.2	80.8	0.0	2.4	4.9	92.7
	42	0.0	2.7	3.4	93.9	0.0	0.0	4.9	95.1
Heavy Tractor Trailer Truck	24	31.5	13.9	19.5	35.1				
	27	32.4	8.7	20.3	38.6				
	32	9.3	16.7	25.0	49.0				
	36	2.8	21.1	19.1	57.0				
	42	0.0	16.3	15.9	67.8				

Table 52. Comparison to Crash Data of RSAPv3 predictions of Penetrating, Rolling over or Vaulting the Bridge Railing for all Vehicle Classes.

Bridge railing height (inches)	BCAP (GSBR) %	ABC (NCHRP 22-08) %	RSAPv3 Full Mix %	RSAPv3 Average Mix %	NCHRP 22-12(3) Crash Data %
24			26.8	20.5	--
27			20.8	14.8	17.1
32	32.7	10.1	15.0	11.9	7.3
36			7.5	3.3	2.4
42			2.8	1.8	0.0

Table 53. Comparison of 1988-1990 TXDOT Bridge Crash Data for bridges built after 1965 with RSAPv3 Predictions for MASH TL3.

Vehicle Type	Vehicle Retained on Bridge		Vehicle Left Bridge	
	TXDOT	RSAPv3 TL3	TXDOT	RSAPv3 TL3
Passenger Car	97.9	99.7	2.2	0.3
Pickup Truck	95.4	95.6	4.7	4.4
Single-Unit Truck	97.7	90.1	2.4	9.9
Tractor-Trailer Truck	92.3	82.7	7.8	17.3

Severity

Once the probability of leaving the roadway and the probability of striking the bridge rail have been calculated, it is necessary to estimate the likely average severity of the crash in order to appropriately apportion the crash costs. RSAPv3 introduced the Equivalent Fatal Crash Cost Ratio (EFCCR) as a measure of crash severity. "$EFCCR_{65}$ is a single, dimensionless measure of crash severity with a particular roadside feature at a baseline speed of 65 mi/hr."[Ray12] The $EFCCR_{65}$ allows for direct comparison of hazard severity between different hazards. The values are based on observable police-reported crashes and adjusted to account for unreported crashes. "Using the $EFCCR_{65}$ to estimate crash severity in a conditional probability model like RSAPv3 provides a systematic methodology based on observed data and established crash severity relationships."[Ray12] This approach removes the subjectivity of previously used crash severity models.

The EFCCR can be considered the probability of a fatal injury crash given that an impact has occurred. $EFCCR_{65}$ values were determined for bridge railings and for vehicles which leave the bridge.

Bridge Railing Crash Severity

An impact with a bridge railing is composed of several possible events each of which has its own associated severity: the impact with the bridge railing itself, the potential for another harmful event if the vehicle is redirected (e.g., rolling over in the roadway or striking another barrier) or leaving the bridge structure and falling to the area below the bridge. This section deals with the first hazard – striking the bridge railing itself.

The development of RSAPv3 included research on the severity of various longitudinal barriers. Table 54 shows an abbreviated list of $EFCCR_{65}$ values, percent of PRVs and percent of impact-side rollover (i.e., RSS) for the longitudinal barriers of interest to this project. The EFCCR is the equivalent fatal crash cost ratio which is the average crash cost divided by the fatal crash cost. If a roadside feature has an EFCCR of 0.0035, for example, and the fatal crash cost is $6,000,000 then the average crash cost on a 65 mi/hr roadway is 0.0035·6,000,000=$21,000. These data were obtained from several data sets as well as from the literature. Ray *et al* provided a summary of data sources in the RSAPv3 Engineer's Manual. [Ray12]

Table 54. $EFCCR_{65}$ of Longitudinal Barriers used in RSAPv3.[after Ray12]

Hazard	**$EFCCR_{65}$**	**%PRV**	**%RSS**
TL3 27" Vertical Wall	0.0098		
TL3 27" NJ SS BR	0.0066	10.08	0.00
TL4 34" Vertical BR	0.0070	0.00	0.00
TL4 34" SS MB	0.0020	0.17	1.01
TL4 32" NJ SS	0.0042	0.06	0.29
TL4 32" F Shape	0.0087	1.38	1.37
TL5 42" Vertical BR	0.0035	0.00	0.00
TL5 42" SS BR	0.0037	0.00	0.00
TL5 42" NJ SS	0.0020	0.15	0.53
TL5 42" F Shape	0.0035	0.67	1.60

The EFCCR is based on observed crashes where the vehicle did <u>not</u> penetrate, over-ride or roll over the feature so it represents the ideal result of a crash (i.e., redirection or stopping in contact with the barrier). For closed-faced concrete barriers, there should be little if any difference in the severity of crashes that result in redirection.

In developing the bridge railing warrants, the $EFCCR_{65}$ used for all closed-face concrete barriers was 0.0035 and the percentage of penetration-rollover-vault (PRV) was set to zero since the penetration and rollover algorithms determine the appropriate value based on the barrier height and vehicle properties. The $EFCCR_{65}$ value of 0.0035 was used to represent the crash severity with all test levels of bridge railings in the development of these selection guidelines. This crash severity applies only to cases where the vehicle is redirected or stops in contact with the barrier. Rollovers that occur during or after redirection are assigned an $EFCCR_{65}$ of 0.0220 consistent with the usual RSAPv3 analysis. The $EFCCR_{65}$ appropriate for use when the vehicle penetrates, rolls over or vaults over the barrier is discussed in the next section.

Bridge Railing Penetration Severity

For purposes of estimating crash severity, the 1989 GSBR and NCHRP 22-08 assumed that penetrating the bridge railing resulted in a 35 ft drop. The subjective severity index (i.e., SI) method was used to rate the severity of the crashes. The 35-ft drop assumption could be modified using an adjustment factor.

RSAPv3 uses a very different severity method based on observed crash data. Striking a bridge railing actually includes several possible outcomes in RSAPv3—the severity of the crash with the bridge railing itself (i.e., discussed in the last section), the possibility of being redirected into another hazard (i.e., striking another barrier or rolling over in the roadway) and penetrating or rolling over a barrier. Penetrating through or rolling over the bridge rail is represented by the edge-of-bridge hazard in RSAPv3. The severity of this hazard is a function of characteristics of the area beneath the bridge. While each bridge rail has a distinct probability of penetration which does not change with the area around the bridge, the possibility of causing harm after the penetration occurs does change after the penetration. For example, a bridge railing on a bridge in a very rural area over a stream will cause no harm to others aside from the occupants of the vehicle. On the other hand, a bridge railing on a bridge over an urban street in a heavily populated area has much more potential for causing harm. This difference in the consequences of penetrating the bridge railing is explained through three types of edge-of-bridge hazards as follows:

HIGH: A high hazard environment below the bridge includes possible interruption to regional transportation facilities (i.e., high-volume highways, transit and commuter rail, etc.) and/or damage to a densely populated area below the bridge. Penetrating the railing may limit or impose severe limitations on the regional transportation network (i.e., interstates, rail, etc.). Penetrating the railing also has the possibility of causing multiple fatalities and injuries in addition to the injuries associated with the vehicle crash itself. Nearby facilities where a collision could lead to a catastrophic loss of life such as chemical plants, nuclear facilities or water supplies should be considered high-hazard environments. A high-hazard environment is also present if penetration or rolling over the bridge railing could lead to the vehicle damaging a critical structural component of the bridge (e.g., a through-truss bridge).

MEDIUM: A medium hazard environment below the bridge includes possible interruption to local transportation facilities, large water bodies used for the shipment of goods or transportation of people, and/or damage to an urban area which is not densely populated (i.e., single family homes, single office buildings, etc.). Penetrating the railing would limit local transportation routes, however, detours would be possible and reasonable. Penetrating the railing has the possibility of causing at least one non-motor vehicle injury or fatality.

LOW: A low hazard environment below the bridge includes water bodies not used for transportation, low-volume transportation facilities, or areas without buildings or houses in the vicinity of the bridge. Penetrating a low hazard railing would have little impact on regional or local transportation facilities. A low hazard railing has no buildings or facilities in the area which present possible non-motor vehicle related victims of a rail penetration.

Bridge rail crash data was examined in Pennsylvania, Ohio, and Nebraska. Only 38 penetrations were found in these censuses of crash data. These 38 bridge rail penetrations provide possibly the only understanding of the consequence of penetrating a bridge rail. The probability of penetration is presented above, however, the severity of a crash which does penetrate the rail was determined from this census of data. While catastrophic penetration events often are news worthy, a census of police reported data should be used to understand the severity of these crashes to remove any bias toward more catastrophic crashes. Zero motorcycles, 26 passenger cars, and 12 heavy vehicles penetrated the bridge rails resulting in five fatal crashes, 13 A injury crashes, 5 B injury crashes, 6 C injury crashes, 8 PDO crashes and 1 crash of unknown severity. This results in an EFCCR of 0.1584. This value was used as the medium level hazard discussed above. A value of 0.0584 was used for the Low-level discussed above. This is equivalent to reducing the K + A crashes to one each. A value of 1 was used for the High-level, which represents absolute certainty that a fatality will be observed every time the rail is penetrated which is consistent with a catastrophic crash.

Table 55. Bridge Railing Penetration Hazard Severities.

Bridge Penetration Severity	$EFCCR_{65}$
Low	0.0584
Medium	0.1584
High	1.0000

The low severity bridge rail penetrations have a severity that is similar to an on-road rollover (i.e., 0.0220) which seems reasonable since in both cases vehicle occupants and the vehicle itself are the only cost components that are at risk.

Costs

When conducting an RSAPv3 analysis, the crash costs of each feasible alternative are determined. A benefit-cost ratio (B/C) for each feasible alternative will be calculated with benefits in the numerator and agency costs in the denominator. Project benefits, in this case, would be defined as a reduction in crash costs between the alternatives under consideration. Project costs include the design, construction, and maintenance costs associated with each alternative.

RSAPv3 determines the crash costs of each user entered roadside design alternative. The three conditional probabilities: (1) the encroachment frequency, (2) the probability of a crash given an encroachment and (3) the probability of an injury given a crash have been discussed above. The results of these analyses are converted to a monetary unit of measure for direct comparison with project costs. The B/C ratio, therefore, is unitless. The benefit-cost ratio (BCR) is defined as follows:

$$BCR_{i/j}=\frac{CC_i-CC_j}{DC_j-DC_i}$$

where: $BCR_{i/j}$ = Incremental BCR of alternative j with respect to Alternative i,

CC_i , CC_j = Annualized crash cost for Alternatives i and j and

DC_i , DC_j = Annualized direct cost for Alternatives i and j.

For each alternative, an average annual crash cost is calculated by summing the expected crash costs for the predicted crashes. These crash costs are then normalized to an annual basis. Any direct costs, as defined by the user (i.e., initial installation and annual maintenance) are also normalized using the project life and the discount rate to an annualized basis and the BCR is calculated. The project life will therefore influence the results, regional variation in agency costs will influence the results and temporal variations in agency costs and crash costs will influence the results. The following sections discuss these influences and the costs used in the development of the selection guidelines.

Project Life

The 1989 GSBR assumed a 30-year life for bridges. The AASHTO Red Book generally recommends a 30-year service life for most transportation projects but the AASHTO LRFD Bridge Design Specification recommends in Section 1.2 a 75-year service life for bridge structures. [AASHTO12, AASHTO07] Replaceable portions of a bridge are sometimes assumed in the AASHTO LRFD specification to have a service life of between 30 and 50 years assuming that the component is either re-furbished or maintained at the end of that period. Bridge railings are certainly a long-lived portion of a bridge structure that would generally only be replaced if the structure is being replaced or if the deck is being replaced or refurbished. Choosing a longer service life amortizes the construction cost over a longer period so higher performance railings would be cost beneficial at lower traffic volumes. Conversely, a shorter service life amortizes the construction cost over a smaller period so higher performance railings would be cost beneficial at higher traffic volumes. In short, a long service life will result in the use of more high performance barriers and a shorter service life will result in proportionally fewer high performance barriers. A design life of 30 years as was used in the development of these selection guidelines, as was done in the AASHTO GSBR.

Regional Cost Variations

Construction Costs

The Washington Department of Transportation (WSDOT) preformed a survey of highway agencies within the United States in 2002 to better understand all project related costs and to gauge how WSDOT costs relate to other States. WSDOT found the average construction cost nationwide was $2.3 Million per lane-mile of highway in 2002. This figure excludes "…right of way, pre-construction environmental compliance, and construction environmental compliance and mitigation." [WSDOT09] These exclusions are quite variable by project and region, let alone State so excluding them allows the comparison to be based only on highway construction costs. Design costs, or the costs related to preparing a project for construction, are generally accepted to be approximately ten percent of the construction costs of the project.

Using the data gathered by the Washington Department of Transportation, the relative cost of each state's construction to the national average can be determined and adjustment factors for regional variations in construction cost can be developed to adjust costs to the national average construction cost. The relative comparison of each responding state to the average value was determined and is shown in Figure 42. States with a value of one have approximately the same construction costs as the national average construction costs, while states with a relative value higher than 1.0 have construction costs which are higher than the average by the multiple shown. For example, New York has a relative cost of 3.6, therefore, it costs more than three times as much to construct a lane mile of highway in New York than the national average. Arkansas and California both have approximately average construction costs, while states like Mississippi, Michigan and Montana have below average construction costs. The regional adjustments vary from a low of 0.44 in Mississippi to a high of 3.63 in New York State. These data show that construction costs reported from different States in the same year vary widely by region.

Crash Costs

Bahar collected eighteen crash costs components from each state to determine the relative crash cost per state. [Baher11] These included the cost of: police, EMS, fire, emergency incident management, Medicaid, coroner, employee medical, employer cost, lost wages excluding taxes, insurance administration, at-fault liability, property damage, legal, court, roadside hardware repair, state tax loss, state welfare safety net, and vocational rehabilitation. This list of cost components is essentially the same as that used by Miller in his 1988 study of nationwide crash costs. [Miller88]

Bahar developed a regional crash cost application tool which includes adjustments for comprehensive crash costs from the national average to each as shown in Table 56.[Baher11] For example, Virginia has a regional crash cost adjustment of 1 indicating crash costs in Virginia are the same as the national average. On the other hand, crash costs in Washington D.C. are 1.61 times higher than the national average and those in Alaska are 64 percent of the national average. Like construction costs, therefore, crash costs also vary widely by region.

Comparison

The regional crash cost adjustment factors from Table 56 are shown alongside the previously introduced construction cost adjustment factors from Figure 42 in Figure 43. Also shown in Figure 43 is the ratio of the adjustment factors for crash cost to construction costs. This adjustment factor ratio is constructed in the same way a benefit/cost ratio would be: the crash cost adjustment is divided by the construction cost adjustment. One might think that these adjustments would generally cancel each other out resulting in a value near unity. In other words, if crash costs are higher in a particular state one might think that the construction costs are higher by the same proportion. In reality, as shown in Figure 43, this is not true. States such as New York, New Jersey and Hawaii have considerably higher construction cost adjustments than crash cost adjustments, while states like Mississippi, New Mexico, and North Carolina have considerably higher crash cost adjustments than construction cost adjustments. A few

States do have equal crash and construction cost ratios (e.g., Massachusetts, Oregon and South Dakota) but they are the exceptions. The ratio of crash costs to construction costs is 36 percent of the national average in New York but twice the national average in Mississippi. Although both crash costs and construction costs vary by region they do not necessarily vary in the same proportion.

As an example, say a national roadside safety guideline was developed using a benefit-cost ratio (BCR) of 2 and national average construction and crash costs. The policy would result in a project whose actual regionally adjusted BCR was $2 \cdot 0.36 = 0.72$ in New York and $2 \cdot 2.02 = 4.04$ in Mississippi. The guideline would have the unintended effect of recommending a non-cost beneficial project in New York and a very cost-beneficial project in Mississippi.

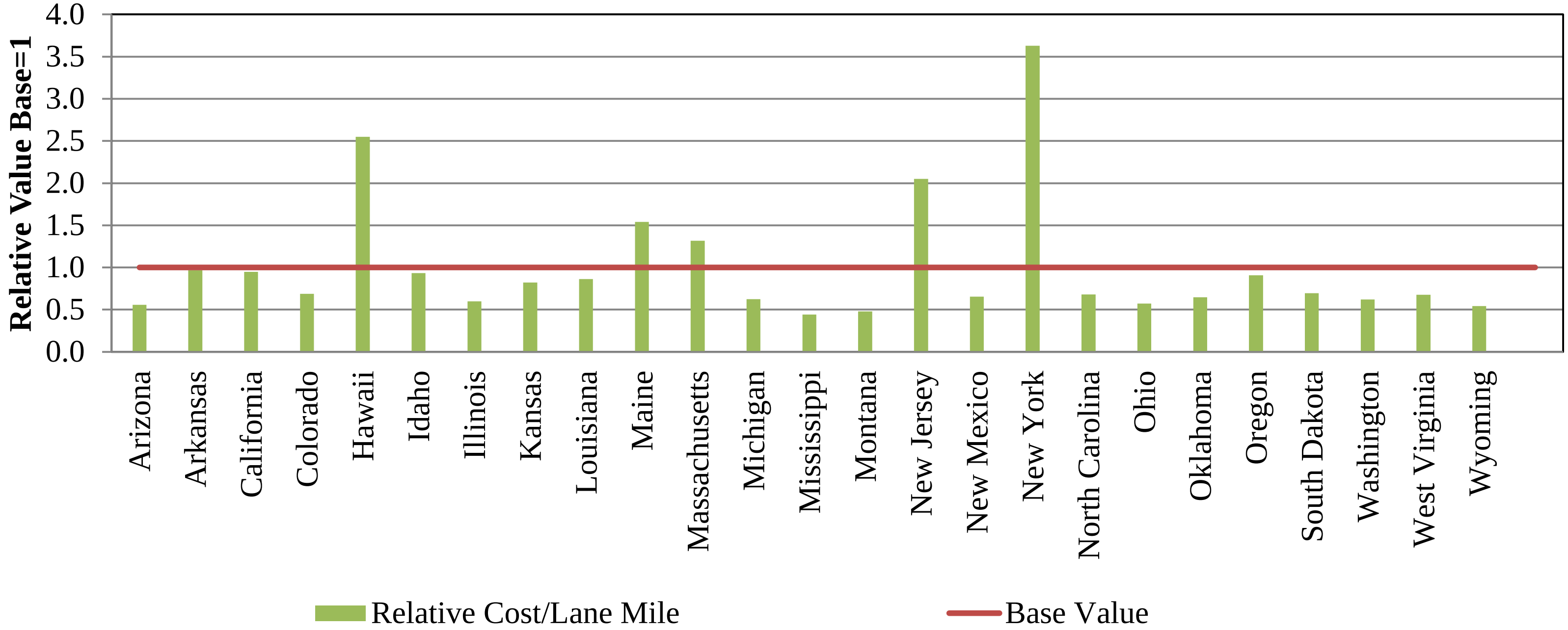


Figure 42. Lane Mile Cost Comparison by State. [after WSDOT09]

Table 56. Crash Cost Adjustments by State to the National Average. [After Bahar11]

State	Adj.	State	Adj.	State	Adj.
Alabama	0.80	Kentucky	0.87	North Dakota	0.88
Alaska	0.64	Louisiana	0.89	Ohio	0.92
Arizona	0.86	Maine	1.08	Oklahoma	0.80
Arkansas	0.91	Maryland	1.10	Oregon	0.87
California	1.23	Massachusetts	1.31	Pennsylvania	1.01
Colorado	0.90	Michigan	0.91	Rhode Island	1.11
Connecticut	1.57	Minnesota	1.22	South Carolina	0.79
Delaware	0.92	Mississippi	0.89	South Dakota	0.73
DC	1.61	Missouri	0.82	Tennessee	0.74
Florida	0.85	Montana	0.79	Texas	0.75
Georgia	0.84	Nebraska	0.94	Utah	0.89
Hawaii	1.56	Nevada	0.96	Vermont	1.10
Idaho	0.93	New Hampshire	0.77	Virginia	1.00
Illinois	0.98	New Jersey	1.28	Washington	1.09
Indiana	0.91	New Mexico	0.89	West Virginia	0.91
Iowa	0.90	New York	1.32	Wisconsin	1.02
Kansas	0.95	North Carolina	0.96	Wyoming	1.04

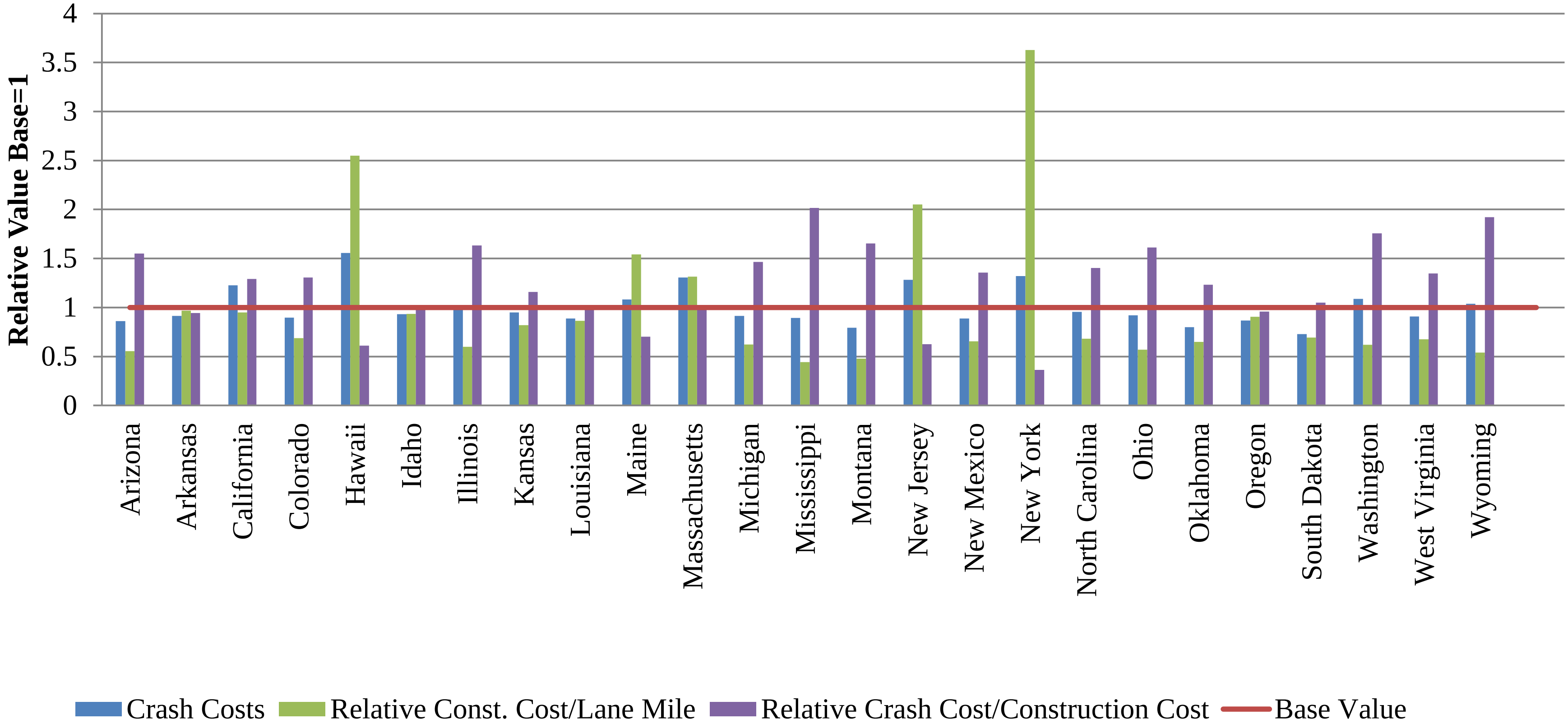


Figure 43. Regional Crash Cost and Construction Cost Adjustment Factors Relative to a Base of One.

Temporal Cost Variations

Further compounding the regional variations in costs are the variations of costs in time due to general economic variations. For example, the U.S. economy was robustly growing in the period between 2003 and 2008 but contracted in 2008 leading to a national recession.

Construction Costs

Since 2003 the FHWA has been collecting highway construction data and using it to calculate the National Highway Construction Cost Index (NHCCI). [NHCCI11] This index can be used to convert or compare current construction expenditures to other years. The NHCCI base of one is relative to the first year of data collection, 2003. When costs increase relative to the 2003 base year, the index is greater than 1 whereas when costs fall below the 2003 year values the index is less than 1. Figure 44 shows the NHCCI index for 2003 through 2012. In 2006, the NHCCI index was 1.35 meaning construction costs were 1.35 times higher than they were in 2003. In 2010, the NHCCI was 1.06 meaning construction costs had decreased almost back to the level of 2003. Figure 44 shows, as would be expected, that highway construction costs decreased by more than 20 percent between 2006 and 2010 in response to the general economic conditions at the time.

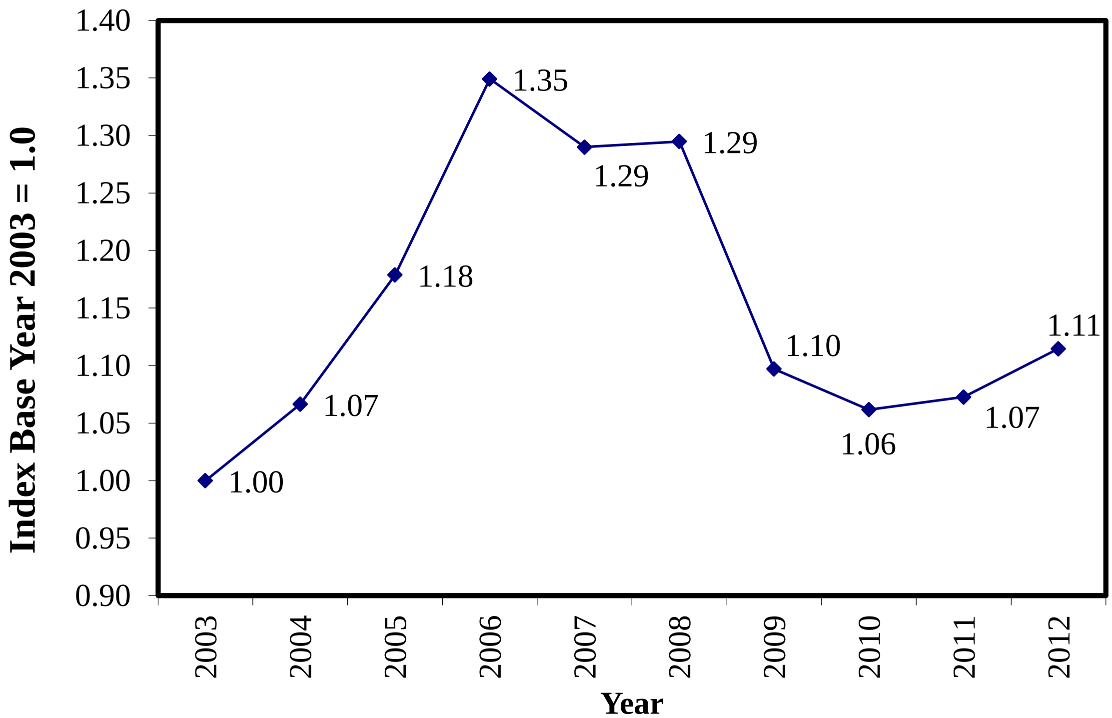


Figure 44. NHCCI Index for 2003 through 2010.

Crash Costs

Miller *et al*. conducted a study in 1988 which determined the comprehensive costs of crashes related to the KABCO scale commonly used on police crash reports to describe the severity of a crash.[Miller88] Each letter of the scale equals a different severity (e.g., K for a fatal injury and O for a property damage only crash) and results in a different comprehensive cost. Miller noted that "these costs should be updated annually using the GDP implicit price deflator."[Miller88] FHWA then updated this study to 1994 dollars. [FHWA09]

FHWA issued a memorandum in 2008 which suggested that the GDP implicit price deflator should no longer be used to update the comprehensive costs of crashes but rather the value of statistical life (VSL) should be used instead. The memorandum notes "the relative values of injuries of varying severity were set as a percentage of the economic value of a life." These values are still being reviewed by FHWA and the relative values may be modified in the future. In 2008, a VSL of $5.8 million was established. In 2009 the VSL was changed to $6.0 million. The value was updated in 2011 to $6.2 million and again in 2012 to $9.1 million. [FHWA08, FHWA09, FHWA11, FHWA12b]

FHWA plans to periodically issue updates to the VSL rather than having users update the comprehensive costs through updates to the GDP, as suggested previously by Miller. Using the relative values of injuries established in 1988 and the 2008, 2009, 2011 and 2012 VSLs provided by FHWA, Table 57 reflects the current comprehensive cost of crashes.

Table 57. Comprehensive Crash Costs by Year.

Crash Severity	Cost per Crash				
	1994	2008	2009	2011	2012
K	$2,600,000	$5,800,000	$6,000,000	$6,200,000	$9,100,000
A	$180,000	$401,538	$415,385	$429,231	$630,000
B	$36,000	$80,308	$83,077	$85,846	$126,000
C	$19,000	$42,385	$43,846	$45,308	$66,500
PDO	$2,000	$4,462	$4,615	$4,769	$7,000

Comparison

After adjusting both the crash and construction indexes to a base year of 2008, the first year the VSL was available, the data can be directly compared as shown in Figure 45. The triangles represent the increasing VSL over the past five years while the diamonds represent the variation in construction costs. While the construction cost varies with general economic conditions, the VSL monotonically increases causing the values to diverge. These diverging values will have tremendous implications to benefit-cost analysis conducted from one year to the next. For example, an alternative which was cost-beneficial in 2010 when construction costs were relatively low may not have been cost-beneficial in 2008 when construction costs were higher. These significant changes can impact the choice of a preferred alternative when using an incremental benefit-cost analysis.

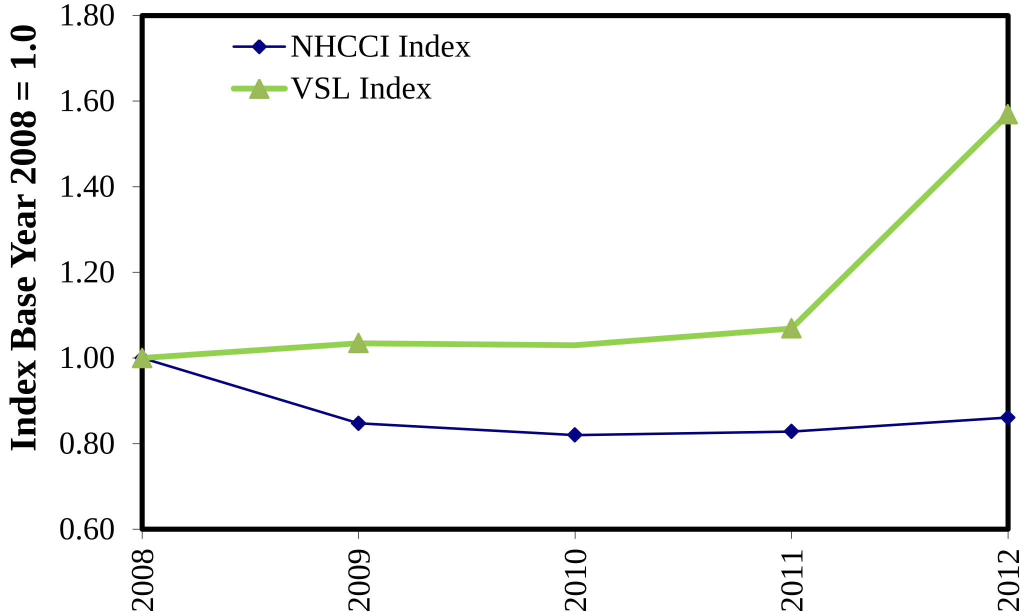


Figure 45. Comparison of Annual VSL to NHCCI Index Updates.

While benefit-cost will always be a valuable tool for choosing among feasible alternatives, the temporal and regional variations of both crash costs and construction costs create a problem when developing national guidelines that are intended for long-term use across all regions of the country. For example, say a roadside design guideline was developed in 2008 that assumed a decision BCR of 2. By 2011 the construction costs would decrease by a factor of 0.82 but the crash cost would have increased by a factor of 1.07 so the benefit-cost of that same alternative would really be 2.6; even better than the conditions of the original policy. On the other hand, if construction prices begin to increase dramatically in the coming years to an NHCCI of 2.6 and crash cost reaches $12 million the same alternative will have a BCR of 1.5, below the threshold value of 2 used to develop the guideline. Since design alternatives generally have design lives of 20 to 30 years, it would seem highly likely that the actual BCR will change dramatically over the life of the project due to temporal and regional variations.

This points out a difficulty of performing benefit-cost analyses during an economic contraction since the real construction costs will decrease but, as a political matter, the value of statistical life is not likely to ever decrease since devaluing the VSL would imply the Federal government places less value on the lives of its citizens. The 2012 values for VSL and NSCCI have been used to adjust costs from different regions and different times to a 2012 national average to develop one option presented in these selection guidelines, however, a risk-based guideline development method which is independent of these temporal and regional variations was explored and ultimately recommended for implementation. The risk-based method is discussed later, but first the costs and adjustments used in the development of the benefit-cost tables are presented.

Bridge Railing Agency Costs

Construction Costs

Construction bid prices for various test level bridge railings from a variety of states were reviewed, adjusted and averaged using the NCHHI index and WSDOT study discussed above to determine the 2012 National average construction prices shown in Table 58. The 1989 AASHTO GSBR assumed the bridge railing construction costs shown on the left side of Table 58. As shown in Table 58, the ratios between the construction cost of each test level bridge railing with respect to TL3 is very similar for both the cost data acquired in this project and that used in the 1989 GSBR. It is also interesting to note that if the 1989 GSBR costs are inflated to 2012 dollars (i.e., 23 years at 4% interest) they are similar to the values obtained in this project from recent construction bid projects in a variety of States so the construction costs appear to be consistent.

The construction cost for the TL2 low-profile concrete barrier and the MASH TL4 concrete barrier are estimates. There have only been a couple of TL2 low-profile barriers built so their documented cost is unreasonably high. In developing the values in Table 58 it is presumed that if the low-profile concrete barrier were built in sufficient volume the cost per linear foot would be just under the cost of a TL3 (i.e., 27" tall) concrete safety shape.

While some crash tests have been performed with the new 36" tall MASH TL4 concrete barriers, there is no known installed inventory so the cost shown in Table 58 was interpolated based on the better documented costs of Report 350 TL4 and TL5 barriers.

Table 58. National Average Construction Costs for Closed-Profile Concrete Bridge Railings.

Performance Level	GSBR Construction Cost $/LF	GSBR Ratio wrt PL1	Equivalent Test Level	2012 Construction Cost $/LF	Ratio wrt TL3
--	--	--	MASH/R350 TL2	100	0.9
PL1	28.80	1.0	MASH/R350 TL3	110	1.0
PL2	43.62	1.5	Report 350 TL4	165	1.5
--	--	--	MASH TL4	240	2.1
PL3	68.96	2.4	MASH/R350TL5	325	2.9

Demolition Costs

A similar review of the cost to remove bridge rail was undertaken. Maryland, Colorado, Oregon and Vermont specifically identify this line item in their standard specifications. Maryland and Colorado list the 2010 weighted average bid price per linear foot as $15 and $6 respectively while Vermont lists the 2011 weighted average bid price as $10.59 per linear foot.

Oregon summarized the 2009 through 2011 weighted average bid prices as $57.23 per linear foot. After applying the state and annual factors then averaging the values, a resulting 2012 value of $23 per linear foot was obtained as the national average cost per linear foot for the removal and disposal of bridge rail. This cost is only applicable when considering the rehabilitation of bridges and upgrading from an existing railing to a new railing.

Repair and Maintenance Costs

Repair costs for bridge railings can be significant but are quite difficult to determine and often not tracked separately. One respondent to the survey conducted during this research suggested an estimate of $54 per linear foot. A review of the Texas and Oregon weighted average bid prices suggests that the 2011 concrete repair price ranges from $65 to $175 per square foot of exposed area, depending on the depth of the repair.

A recent project from the State of Florida to enhance its bridge management software includes cost elements for all types of bridge construction and repair. [Sobanjo11] The data contained 176 winning bids for work on bridge barrier repairs and retrofits. The 2009 repair of railing line item ranged from $215 to $3,880 per each event, however, the railing type or material was not specified.

Many States prefer concrete bridge railings because for the vast majority of crashes there will be little important barrier damage. In the rare case of a major collision involving a heavy vehicle, however, concrete bridge railings can fail catastrophically. Such catastrophic damage may including the complete structural failure of the concrete barrier itself as well as failure of the bridge deck since many concrete bridge railings are constructed integral with the deck. It appears that a reasonable price for minor repairs would be $110 per square foot.

Bridge Railing Crash Costs

The comprehensive costs of all vehicle crashes is discussed above and shown in Table 57 for 1994 through 2012. Zaloshnja and Miller determined the comprehensive cost of various truck crashes by truck size. [Zaloshnja06] These costs were reported by total crash cost by most severe injury and by cost of injury per victim. These costs include the following categories: "(1) medically related, (2) emergency services, (3) lost productivity (wage and household work), and (4) the monetized value of pain, suffering, and lost quality of life." [Zaloshnja06] A summary of the findings are presented in Table 59. Table 59 includes the annual number of truck crashes by crash severity and truck type as well as the comprehensive cost of truck crashes by crash severity and truck size.

Table 59. Annual Number and 2005 Cost of Truck Crash by Injury Severity and Truck Type. [After Zaloshnja06]

Truck Type	Max Severity in Crash	Annual Number of Crashes	Cost per Crash
Straight truck, no trailer	K	1,016	$3,136,409
	A	2,612	$640,494
	B	4,665	$198,225
	C	17,491	$62,364
	O	116,476	$13,286
	U*	527	$44,307
	Unk	7,245	$22,114
Straight truck with trailer	K	162	$3,142,831
	A	594	$363,436
	B	517	$220,440
	C	1,359	$91,530
	O	12,502	$17,295
	U*	20	$45,990
	Unk	1,277	$23,396
Bobtail	K	37	$3,172,568
	A	858	$381,348
	B	266	$173,507
	C	1,269	$64,324
	O	9,843	$19,089
	U*	59	$22,923
	Unk	786	$22,401
Truck-tractor With one trailer	K	2,825	$3,833,721
	A	10,843	$437,845
	B	17,688	$171,710
	C	19,461	$90,959
	O	179,181	$15,749
	U*	413	$33,397
	Unk	10,191	$24,939
Truck-tractor with 2 or 3 trailers	K	150	$3,352,753
	A	1,129	$121,936
	B	559	$244,084
	C	740	$116,920
	O	4,976	$24,883
	U*	-	-
	Unk	420	$30,872
Medium/ heavy truck	K	87	$3,105,969
	A	-	-
	B	259	$235,327
	C	455	$78,442
	O	3,143	$10,072
	U*	6	$34,734
	Unk	1,767	$19,435
All medium/ heavy trucks	K	4,278	$3,604,518
	A	16,035	$525,189
	B	23,955	$180,323
	C	40,774	$78,215
	O	326,121	$15,114
	U*	1,024	$38,661
	Unk	21,685	$23,479

U*-Injury, unknown severity
Unk-unknown severity

The FMCSA updated these costs to reflect the FHWA 2008 updated VSL. These updated 2008 costs are shown in Table 60. The cost of a fatal truck crash in 2005 was approximately $3.6M, while the 2008 estimate jumped to $7.2M which is a reflection of the increase in the statistical value of life. [FMCSA08] Knowing the relationship between the cost per crash and the cost per victim in 2005, the relationship between the 2005 cost per crash and the 2008 cost per crash, along with the published VSLs for 2008 and 2012, an estimate of the 2012 crash cost per victim was calculated. This information is also shown in Table 60.

Table 60. 2005 Cost of All Truck Crashes by Injury Severity and per Victim. [After Zaloshnja06]

Truck Type	Max Severity in Crash	Annual Number of Crashes	2005 Cost per crash	2005 Cost per Victim	2008 Cost per Crash	2012 Cost per Victim
All medium/ heavy trucks	K	4,278	$3,604,518	$3,055,232	$7,200,310	$9,575,482
	A	16,035	$525,189	$325,557	$1,049,107	$1,020,746
	B	23,955	$180,323	$134,579	$360,209	$421,788
	C	40,774	$78,215	$62,702	$156,241	$196,516
	O	326,121	$15,114	$5,869	$30,191	$18,395
	U*	1,024	$38,661	$33,759	$77,228	$105,809
	Unknown	21,685	$23,479	$20,540	$46,901	$64,375

Using the information from Zaloshnja and Miller an approximate comprehensive cost of the crashes investigated by NTSB or reported in the media discussed earlier in this report can be determined. [Zaloshnja06] These costs are presented to show the possible range of crash costs for these catastrophic bridge rail crashes. This comprehensive cost data is appropriate for cost/benefit analysis when evaluating the cost of crashes that might happen against the cost of safety improvements. However, the crash cost values published by the National Safety Council (NSC) are best suited for calculating the cost of crashes which have already happened. The NSC values can be used to measure the economic loss (i.e., economic impact) of crashes. The NSC values do not include what people are willing to pay for improved safety. [NSC11] The NSC values are therefore lower than the comprehensive cost values. The NSC crash costs per victim values for 2011, as well as the inflation adjusted calculated values for 2012 are shown in Table 61. Using these values and the comprehensive crash cost values, the costs of each NTSB investigated or media reported crash was determined and is shown in Table 62 and Table 63.

Table 61. NSC Economic Impact Crash Costs.[NSC11]

Severity	2011 Cost /Victim	2012 Cost/Victim
Death	$1,420,000	$1,449,386
Nonfatal Disabling Injury	$78,700	$80,329
Property Damage Crash (including non-disabling injuries)	$9,100	$9,288

Table 62. Summary of the Cost of Bridge Rail Crashes in the Media.

Crash	2012 Comprehensive Crash Costs	NSC 2012 Crash Cost	Additional Unknown Costs
St. Petersburg, Florida, 2001	$9,823,890	$1,467,962	
Glenmont, New York, 2007	$18,395	$9,288	
Wiehlthal Bridge, Germany, 2004	$9,575,482	$1,449,386	$400 Million in Repairs to Structure
Boston, Massachusetts, 2007	$843,576	$18,576	
San Francisco, California, 2009	$9,575,482	$1,449,386	
Amesbury, Massachusetts, 2011	$9,575,482	$1,449,386	
Avon, Colorado, 2012	$9,612,272	$1,467,962	
Syracuse, New York, 2012	$843,576	$160,658	
Montreal, Quebec, 2011	$19,150,964	$2,898,772	
Avellino, Italy, 2013	$364,926,406	$55,169,548	
Beaverton, Oregon, 2012	$421,788	$9,288	
Bronx, New York, 2012	$67,028,374	$10,145,702	
Grand Prairie, Texas, 2013	$9,575,482	$1,449,386	
Boston, Massachusetts, 2013 (1)	$214,911	$9,288	
Boston, Massachusetts, 2013 (2)	$18,395	$9,288	
Buellton, California, 2012	$11,018,016	$1,539,003	
Galesburg, Illinois, 2013	$9,681,291	$1,458,674	
Williamsburg, Kansas, 2012	$51,082,801	$7,509,756	

Table 63. Summary of the Cost of Bridge Rail Crashes Investigated by NTSB.

Crash	2012 Comprehensive Crash Costs	NSC 2012 Crash Cost	Additional Unknown Costs
Fort Sumner, New Mexico, 1972	$183,521,293	$27,677,654	
Nashville, Tennessee, 1973	$76,603,856	$11,595,088	
Siloam, North Carolina, 1975	$45,050,536	$5,946,152	
Martinez, California, 1976	$277,688,978	$42,032,194	New bridge constructed
Houston, Texas, 1976	$76,286,894	$10,349,580	Destroyed 94 feet of bridge railing, damaged bridge deck, and column supporting overpass was sheared off.
Elkridge, Maryland, 2004	$38,301,928	$5,797,544	
Huntsville, Alabama, 2006	$62,825,006	$7,321,033	
Sherman, Texas, 2008	$167,824,682	$25,072,835	

The costs of repairs to bridges are considerably higher than any other given section of highway and generally not considered in the comprehensive and NSC crash cost figures presented above. In addition to these calculated crash cost values, for example, the bridge rail crash in Wiehlthal Germany ultimately required that the entire bridge be reconstructed. [Wiehlthal04] The temporary traffic control and bridge repair costs totaled approximately $400 million. The bridge rail crash in Martinez, California [Marinez76] required that a new bridge be constructed in addition to the approximately $277 million in calculated crash costs.

Sight Distance Considerations

The AASHTO LRFD Bridge Design Specification (LRFD) states in section C2.3.2.2.2 "Special conditions, such as curved alignment, impeded visibility, etc.., may justify barrier protection, even with low design velocities."

The AASHTO Policy on Geometric Design of Highways (Green Book) offers this guidance for sight distance considerations:

> "For *stopping sight distance* calculations, the height of object is considered to be 600 mm [2.0 ft] above the road surface. … The basis for selection of a 600 mm [2.0 ft] object height was largely an arbitrary rationalization of the size of object that might potentially be encountered in the road and of a driver's ability to perceive and react to such situations." [AASHTO01]
>
> For *passing sight distance* calculations, the height of object is considered to be 1,080mm [3.5 ft] above the road surface." "It is not necessary to consider passing sight

distance on highways or streets that have two or more traffic lanes in each direction of travel." [AASHTO01]

When "horizontal sight distance on the inside of a curve is limited by obstructions" measurements are taken at an average of the stopping sight distance and passing sight distance height (i.e., 2.75 ft). "Such refinement on two-lane highways generally is not necessary and measurement of sight distance along the centerline of traveled way edge is suitable." [AASHTO01]

For *Intersection Sight Distance*, "the determination of whether an object constitutes a sight obstruction should consider both the horizontal and vertical alignment of both intersecting roadways, as well as the height and position of the object." In making this determination, it should be assumed that the driver's eye is 1,080mm [3.5ft] above the roadway surface and that the object to be seen is 1,080mm [3.5ft] above the surface of the intersecting road. "The recommended dimensions of the sight triangles vary with the type of traffic control used at an intersection…." [AASHTO01]

It seems the LRFD and Green Book have somewhat conflicting guidance for improving visibility along the roadway, where the LRFD suggests considering a barrier if visibility is limited and the Green Book limits object height to two feet (24 inches) for stopping sight distance (SSD), 3.5 feet (42 inches) for passing sight distance (PSD) and 2.75 feet (33 inches) for horizontal sight distance (HSD).

All TL2 through TL5 bridge railings are 24 inches high or taller so even a TL2 bridge railing is a sight obstruction for stopping sight distance since the obstruction height is 24 inches. The guidelines presented herein use a shoulder width of eight feet so if, as suggested by the AASHTO Green Book, the sight line is taken from the "centerline of the travelled way edge" and the lane width is assumed to be 12 feet, then the middle ordinate of the circular curve is 20 feet (i.e., 12+8=20). Knowing the stopping sight distance based on the design speed and the middle ordinate, the minimum radius of the horizontal curve which would not present a stopping sight distance obstruction was calculated and is shown in Figure 46 and Table 64. Also shown in Figure 51 is the Green Book recommended minimum horizontal curvature based on 10 percent super-elevation and design speed. As shown in Figure 46 and Table 64, the minimum radius imposed by the stopping sight distance obstruction of the barrier is always greater than the minimum Green Book radius from Green Book Exhibit 3-14 and the disparity increases with the design speed. This is true even if the shoulder width is increased to 14 feet or more which is probably an unreasonable shoulder width to expect on a typical bridge. Stopping sight distance is generally considered necessary at all points along the highway but the above analysis suggests that it is not possible to supply the appropriate stopping sight distance for radii less than the dashed line in Figure 51 with <u>any</u> bridge railing.

The same curves apply for "horizontal sight distance" (HSD) and indicate that MASH TL4 and TL5 bridge railings would limit the HSD when the radius of the curve is less than the dashed line in Figure 46. If HSD were considered in the bridge rail selection then the values shown below in Table 64 could be used as minimum radii. The Green Book recommendation for the HSD object height is 33 inches.

Intersection sight distance (ISD) can be impeded by bridge rail choice. Options may exist early in design to realign the intersection to provide an acceptable sight triangle. There are plans to study improving roadside safety hardware choices at bridge ends/intersection in the near future that may provide some additional options and insight to this problem (i.e., NCHRP Project 15-53). The Green Book recognizes the continual development of roadside hardware through a single statement: "highway designers should recognize the dynamic developments currently under way in the entire area of roadside design. … Highway designers should endeavor to use the most current acceptable information in their designs." Maintaining the intersection sight triangles may necessitate stopping or modifying the barrier as the end of the ramp or bridge is approached.

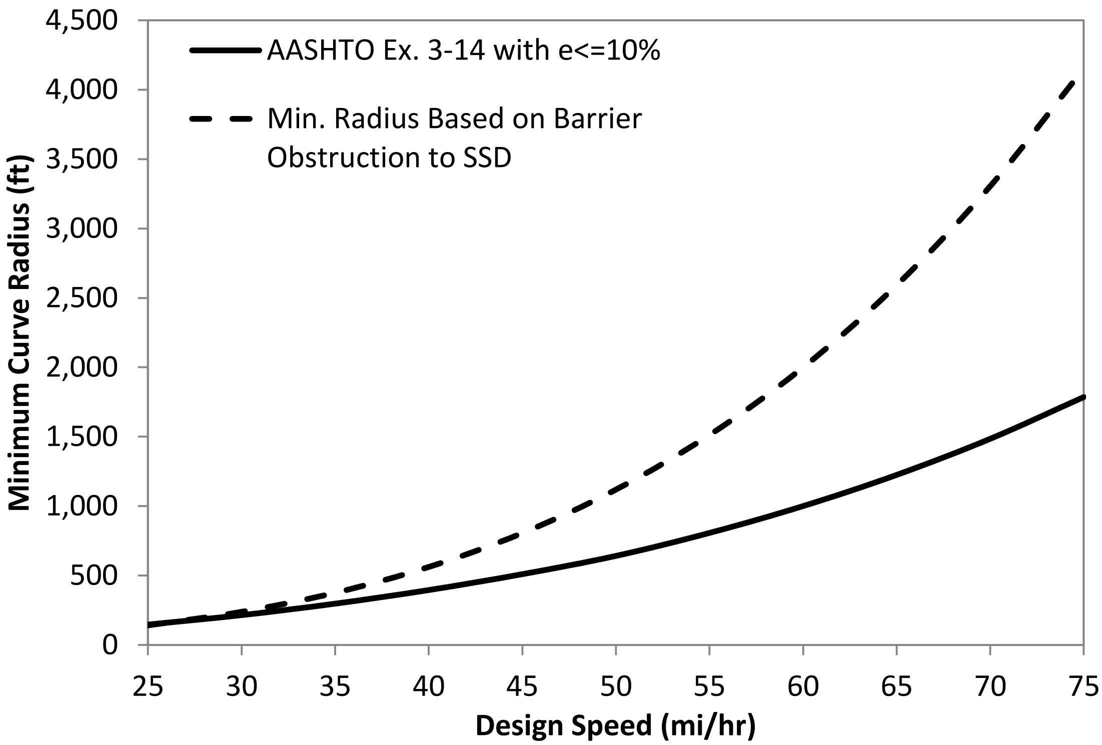


Figure 46. Minimum Horizontal Curve Radius Based on Barrier Obstruction to the Stopping Sight Distance Compared to AASHTO Exhibit 3-14.

Table 64. Minimum Radius and Maximum Degree of Curve Based on the Horizontal Sight Distance Obstruction from a TL4 or TL5 Bridge Railing.

Design Speed	Minimum Radius	Maximum Degree of Curve
25	141	40.7
30	238	24.0
35	375	15.3
40	561	10.2
45	805	7.1
50	1,119	5.1
55	1,512	3.8
60	1,999	2.9
65	2,592	2.2
70	3,305	1.7
75	4,153	1.4

Analysis Methods

The objective of this project was to establish selection guidelines for test level two through five bridge railings. The publication of MASH at the onset of this research required some evaluation of which crash testing procedures the barriers guidelines should be based on. A decision was made to limit the selection guidelines to MASH performance criteria, thereby having this research completely coordinated with the current literature on crash testing bridge railings. The alternatives considered in the analyses were, therefore, MASH TL2 through MASH TL5.

The analyses performed to develop the selection procedures were performed using RSAPv3 release 130912XL14. [Ray12] RSAPv3 is the software implementation of a conditional probability model for estimating the number and severity of crashes based on roadway, traffic and site conditions. The model has three basic parts:

- Estimating the number of vehicle encroachments (i.e., the encroachment module),
- Estimating the probability of a vehicle striking a hazard if it leaves the roadway (i.e., the collision module) and
- Estimating the expected crash cost or severity of a collision with a roadside object if it occurs (i.e., the severity module).

Details on the methods and algorithms used in RSAPv3 functions are available elsewhere and will not be repeated here (see [Ray12]) other than those specifically related to or updated for this project. The inputs to the conditional probability model and necessary changes to RSAPv3 needed to accomplish the goals of this project include determining:

- Encroachment characteristics for heavy vehicles,

- Encroachment characteristics over the project life at low traffic volumes (i.e., averaging out the Cooper data "humps"),
- Vehicle mix characteristics,
- Penetration, rollover, and vault potential for each test level of bridge railing,
- The severity of a bridge rail penetration if it does occur, and
- An improved understanding of how temporal and regional variations in crash costs and construction costs impact the development of national guidance development.

Each of these issues has been discussed in detail in an earlier section of this report. Using these inputs and having made the necessary changes to RSAPv3, the four alternatives (i.e., MASH TL2 through TL5) were evaluated across a range of traffic volumes and heavy vehicle percentages to determine the expected annual crash cost of each alternative. Using the expected annual crash costs, a benefit-cost approach or a risk approach can be taken to generate the selection guidelines.

Benefit-Cost versus Risk Approach

There are advantages and disadvantages to both methods of analysis. The benefit-cost method has the advantage that it includes both societal benefits (i.e., reduction in crash costs) and agency costs (i.e., construction, maintenance and repair) such that the benefits are maximized while making the best possible use of agency funds. The disadvantage is that since costs are explicitly included, regional and temporal variations in the cost elements can make the same solution cost-beneficial in one region and not cost beneficial in another. Similarly, an alternative that is cost beneficial under one set of economic assumptions may become not cost beneficial if economic conditions change in the future. Another disadvantage to a benefit-cost approach is that the risk is not necessarily uniform so one cost-beneficial solution can have a different inherent risk than another with the same benefit-cost ratio.

On the other hand, risk analysis sets a specific risk objective that is uniform across regions and through time such that the risk of an unacceptable event is always the same. Temporal and regional variations in either the crash or construction cost do not change the underlying risk of each alternative. The disadvantage is that the best risk-based solution may not always be cost-beneficial in every region or at every point in time.

The primary advantage to a risk-based approach is that construction and maintenance costs do not affect the results so the performance goal will not change over time or from one region to another. This allows the policy decision about the appropriate level of safety that should be provided to be separated from the question of which alternative is the most economically attractive. A further advantage is that policy decisions for new construction, rehabilitation, or retrofitting would be identical using a risk-based approach. For example, a decision to install 32-inch concrete bridge rail for 20,000 vpd and 10% trucks would be set regardless of whether the barrier will be installed where no barrier currently exists (i.e., new barrier construction costs only) or for replacing an existing barrier (i.e., new barrier construction costs plus demolition and removal of an existing barrier costs). The risk-based goal remains the same for both problems.

Benefit-cost analysis has been a valuable tool in roadside safety for nearly 35 years and has been used to both prioritize specific projects as well as develop State and national guidelines for barrier selection and placement. One the one hand, benefit-cost analyses helps transportation agencies make the most effective use of their limited roadside safety funding. On the other hand, the level of risk implicit in these decisions is usually hidden, resulting in different levels of risk based on the time an analysis was performed and the region where the alternative was placed in service.

The two tools can, in fact, be used together when guidelines are developed based on acceptable risk criteria. Benefit-cost methods can be used to determine the most economical way of achieving the desired risk level on a project-by-project basis. Agencies in different regions may chose different roadside safety alternatives based on the economic situation in their locale but the overall level of risk would be uniform throughout the nation. For these reasons, a risk method has been used in the development of the selection guidelines.

While the selection guidelines are presented as a risk-based approach, the process through which the guidelines are applied is very flexible. Additional tables have been provided in Appendix A for those that prefer a cost-benefit approach. Both sets of tables and the selection process are discussed below.

Developing the Selection Guidelines

The most straight-forward way to develop selection guidelines would be to simply run a large number of RSAPv3 analyses and present them in table format with the construction year AADT listed in the rows and percent trucks in the columns and each entry indicating the appropriate test level. For three different highway types and three hazard environments this would have resulted in at least nine pages of tables not including additional material needed for the roadway adjustment factors like horizontal curvature and grade as well as space to explain the procedure. Adding a dozen pages to Chapter 13 of the LRFD Bridge Design Manual for bridge railing selection did not seem to be the best approach so another more concise method was developed as discussed below.

The encroachment-based procedure used by RSAPv3, as discussed above, is naturally broken into three parts: probability of encroachment, probability of collision and expected severity. For bridge railings, the bridge railing is located relatively close to the edge of the travelled lane (i.e., shoulders are generally between two and 12 ft) so the probability of collision estimation in a bridge railing problem is fairly simple and is easily combined with the severity portion of the analysis. The analysis can be deconstructed into two parts – encroachment and collision/severity – such that the guidelines can be made much more concise.

The problem of estimating the risk associated with observing a severe or fatal crash along a 1000-ft segment of bridge railing over its 30-year life is analogous to a coin toss problem and, therefore, follows the binomial distribution:

$$P(k) = \left[\frac{n!}{k!\,(n-k)!}\right] p^k (1-p)^{n-k}$$

where,

P(k)	=	Probability or risk of observing k failures (i.e., severe or fatal crashes) in n trials,
n	=	Number of trials (i.e., the number of encroachments over the life of the bridge
k	=	Number of failures (i.e., severe or fatal crashes) observed and
p	=	Probability of a failure in any one trial.

The equation above shows that if a fair coin (i.e., a fair coin is one where the probability of "tails" = probability of "heads" = 0.5) is tossed 10 times, the probability that "heads" will appear twice is:

$$P(k) = \left[\frac{n!}{k!\,(n-k)!}\right] p^k (1-p)^{n-k} = \left[\frac{10!}{2!\,(10-2)!}\right] 0.5^2 (1-0.5)^{10-2} = 0.0440$$

The chance of observing exactly two "heads" in a series of 10 fair coin tosses is 0.0440 or approximately 1 in 22 (i.e., 1/22~0.0440). Stated another way, if 22 people flip a coin ten times, one of those persons should get two heads and eight tails and the other 21 will get some other combination. While the probability of observing exactly two heads is very small, it is not zero.

In the case of a severe or fatal crash involving a bridge railing, the number of trials is the number of encroachments expected over the 30-year life of the bridge railing and the number of failures is the number of severe or fatal crashes that should occur in that time period.

The term on the left hand side of the equation, P(k), is the risk of observing one or more crashes with a particular severity over the life of the bridge railing. Only three values are required to calculate the risk of a severe or fatal crash over the life of a bridge railing in this problem:

n	=	The number of encroachments over the 30-year life of 1,000-ft of bridge railing,
k	=	1 and
p	=	Probability of a severe or fatal crash in any encroachment.

Estimate the number of life-time encroachments on 1,000-ft of bridge railing

Input to the guidelines are the construction year AADT, anticipated traffic growth and percent of trucks in the traffic stream as well as roadway characteristics like the highway type, speed limit, horizontal curvature, grade, lane width and access density. The first step in the analysis, therefore, is to predict the number of encroachments that can be expected over the life of the bridge railing based on the highway type and AADT. As discussed earlier, the assumed life of a bridge railing is 30 years and a traffic growth of 2 percent per year was assumed in developing the guidelines.

Bridges, of course, vary in length so it is necessary to select a standard unit length for the analyses. A unit length of 1,000-ft was chosen. Any length could have been used to develop the selection guidelines but the same length needs to used throughout the process since the risk criterion includes units of length.

RSAPv3 uses the so-called Cooper data to estimate the number of encroachments as a function of highway type and AADT. The Cooper data, which was collected over a wide area in Canada in the 1970's, along with the statistical methods used to develop the relationships pictured in Figure 47 are discussed in much more detail in the RSAPv3 Engineer's Manual.[Ray12]

One of the interesting features of the Cooper data shown in Figure 47 are the pronounced "humps" in each of the curves. For two-lane undivided highways, the top of the "hump" occurs at about 5,000 veh/day and for four-lane divided highways it occurs at about 30,000 veh/day. The relationship is a little clearer if the encroachment rate (i.e., encroachments per million vehicle miles travelled) is examined as is shown in Figure 48. The encroachment rate is highest for two-lane undivided roadways at low volumes. The encroachment rate for all highway types decays rapidly as the lane volume increases. Eventually each curve reaches an asymptotic value; The two-lane undivided curve reaches the value of 0.0448 encroachments/MVMT for all lane volumes greater than about 8,000 veh/day and the four-lane divided curves reaches the value of 0.1160 encroachments/MVMT for all values greater than about 10,000 veh/day.

Some may think the relationships shown in Figure 47 are counter-intuitive since four-lane divided highways have a larger number of expected encroachments than two-lane undivided highways at the same traffic volume regardless of whether rates or frequencies are examined. One important fact to remember in viewing Figure 46 and Figure 47 is that the Cooper data predicts encroachments not crashes. While the observational Cooper data suggests that more vehicles encroach on the roadside on four-lane undivided highways, the roadside of four-lane divided highways generally feature much more generous clearzones, roadsides with less dramatic slopes and many fewer fixed objects so while there may be more encroachments, there are fewer crashes.

In developing the selection guidelines, RSAPv3 analyses were performed for the following conditions:

- Two-lane undivided, four-lane divided and one-way highway types,
- 1,000-ft long highway segment,
- 500 veh/day ≤ AADT ≤ 110,000 veh/day,
- 30 year design life,
- 2 percent annual traffic growth and
- Flat, tangent sections with 12-foot lanes and 8-ft shoulders.

The results were then tabulated by AADT and highway type as shown later in Figure 50 and Table 68.

At this point, the adjustments for roadway conditions like grade, horizontal curvature, lane width, access density, number of lanes and posted speed can be determined. The adjustments are applied to the number of expected encroachments. The result of this first step, then, is a tabulation of the expected number of encroachments over the 30 year life assuming a 2-percent growth that also accounts for the roadway characteristics at the site.

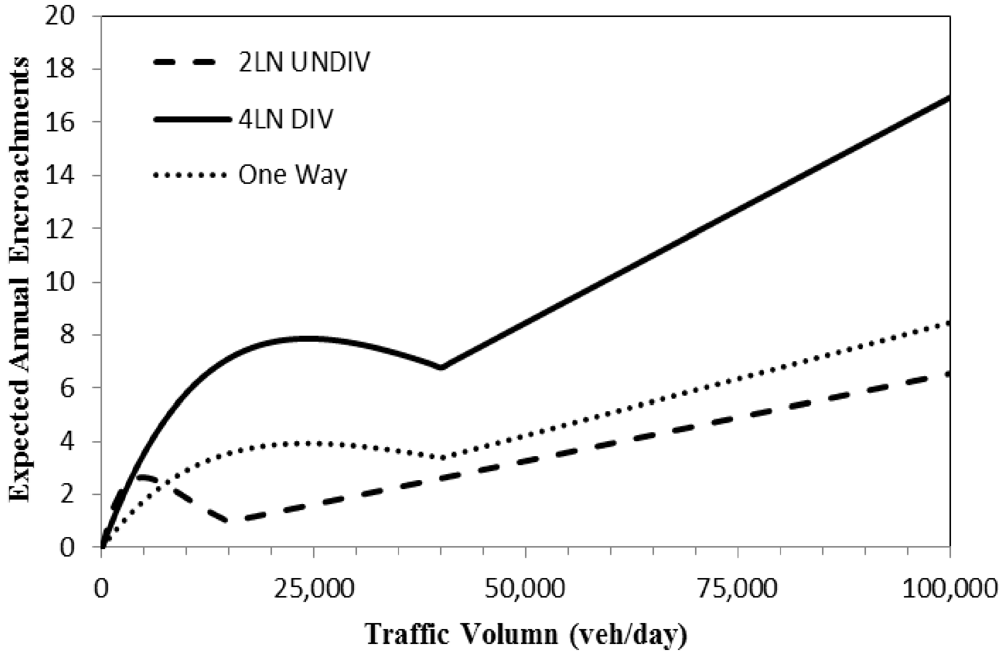


Figure 47. Cooper Encroachment Frequency Data [after Ray12]

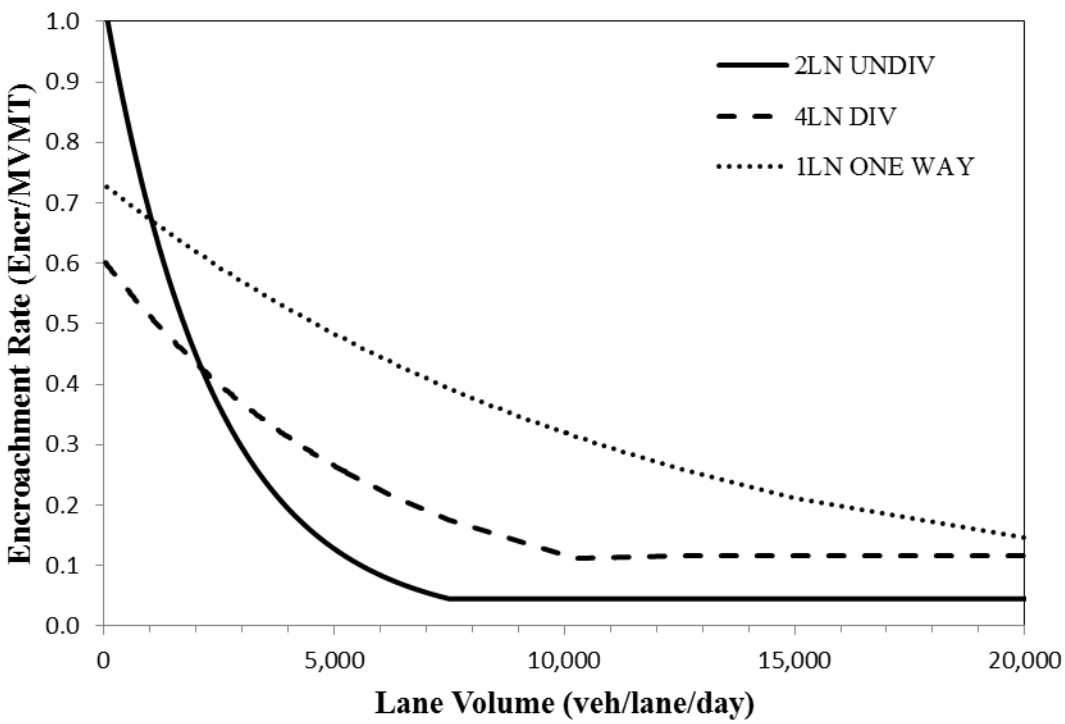


Figure 48. Cooper Encroachment Rate by Lane Volume. [after Ray12]

Estimate the probability of a severe or fatal crash in any encroachment

The probability of a severe or fatal crash occurring in each particular collision is the next parameter that needs to be calculated for applying the binomial distribution. RSAPv3 uses the conventional police reported crash severity (i.e., the KABCO scale) which can be converted into dollar values based on the FHWA crash cost recommendations for the particular year of interest. While the fatal crash cost (i.e., the value of statistical life) changes from year to year as discussed earlier, the distribution of crash costs by severity does not; the proportion of each injury severity with respect to a fatal crash remains constant based on the work of Miller. [Miller88] Table 65 shows a table of crash costs by police reported crash severity and the EFCCR represented by each severity. As shown in Table 65, an encroachment whose sum of event EFCCRs is equal to 0.0692 corresponds to a severe (i.e., A-level in the police reported scale) injury so any encroachment resulting in an EFCCR (i.e., dimensionless crash cost) of 0.0692 or greater is a severe or fatal crash (i.e., A+K).

Table 65. Police Reported Crash Severity by Crash Cost and EFCCR.

	2006 Crash Cost	EFCCR
K	$ 2,600,000	1.0000
A	$ 180,000	0.0692
B	$ 36,000	0.0139
C	$ 19,000	0.0073
PDO	$ 2,000	0.0008

For each encroachment, RSAPv3 calculates the cumulative EFCCR for each event that occurs during the encroachment. The EFCCR is generally converted to a crash cost considering the VSL and vehicle type for each predicted crash. When only considering the risk of a crash, the EFCCR of each encroachment is not converted to crash costs, but is carried forward as an EFCCR and the cumulative probability distribution of EFCCRs is obtained similar to the example shown in Figure 49.

Using the cumulative crash severity distribution in Figure 49 for a TL2 bridge railing in a low-hazard environment, an EFCCR of 0.0692 is associated with a probability of 0.9560. The probability of an EFCCR of 0.0692 or greater (i.e., the probability of a severe or fatal injury) is 1 – 0.9560 = 0.0440. If the hazard environment is high, an EFCCR of 0.0692 corresponds to a cumulative probability of 0.8552 for a TL2 bridge railing and 40 percent trucks. For a high-hazard environment, therefore, the probability of a severe or fatal injury crash on a TL2 bridge railing is 1 – 0.8552 = 0.1448. A cumulative probability distribution of the encroachment EFCCRs was developed for each test level bridge railing, each hazard environment and each percent of trucks and the cumulative probability of a severe or fatal crash was assembled in Table 65. Each entry in Table 66 is associated with a cumulative probability distribution like the example shown in Figure 49. The cumulative probability distributions, like the examples shown in Figure 49, are the conditional probability of a severe of fatal injury involving a particular test level bridge railing given that a crash occurs so they are only dependent on the vehicle mix (i.e., percent of trucks) and the hazard environment. This fact is what allows the guidelines to be split into an encroachment prediction part and a crash severity prediction part.

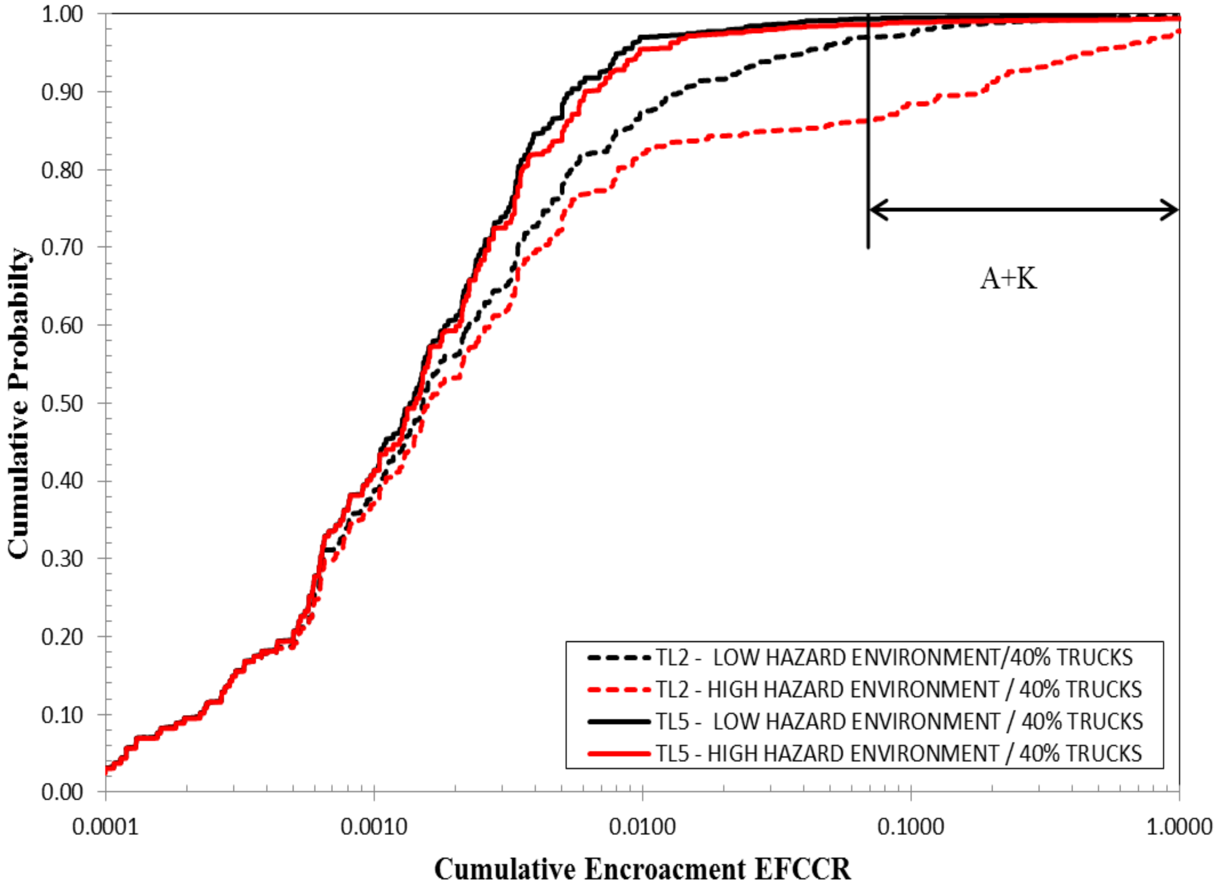


Figure 49. Cumulative EFCCR Distribution for TL2 and TL5 Bridge Railings with 40% Trucks.

Unlike the fair coin toss discussed above where the probability of "failure" is 0.5, the probability of observing a failure in a bridge rail crash is very small and depends on (1) the number of trucks in the traffic mix and (2) the test level of the bridge railing. As shown in Table 66, for a given hazard environment, the probability of single encroachment resulting in a severe or fatal crash decreases as the test level increases. This would be expected with an increase in the performance level of the bridge railing. Also, for a particular test level bridge railing, the probability of a severe or fatal crash increases as the percent of trucks increases which is also as expected since more heavy vehicles in the mix make a collision with a very heavy high-energy vehicle more likely.

The values in Table 66 are invariants of AADT and encroachment meaning that the probability of a single encroachment resulting in a severe or fatal crash does not depend on the traffic volume or the roadway characteristics. The AADT, traffic mix and roadway characteristics influence the number of encroachments to be expected but not the severity of any particular crash. The severity is influenced by the area under the bridge (i.e., High, Medium, Low). Thus, Table 66 only needs to be generated once.

The probabilities shown in Table 66 where obtained by performing RSAPv3 analyses at varying percentages of truck traffic for the following conditions:

- Primary right encroachments only,
- Shoulder width of 8 ft (i.e., face of bridge railing is 8 ft from the edge of the lane),
- 65 mi/hr divided highways and
- An encroachment increment of 500 ft on a 1,000-ft segment of roadway (Station 5+00 to 15+00) in the middle of a 2,000 section of bridge railing (Station 0+00 to 20+00).

Calculating the risk

Now that the number of encroachments and the probability of a single crash resulting in a severe or fatal crash have been estimated, the risk of observing a severe or fatal crash over the 30-year life of a 1,000-ft section of bridge railing can be calculated. The risk can now be calculated using the cumulative density function of the binomial distribution with:

k = The number of failures =1,
n = The expected number of encroachments from the first step and;
p = The probability of any particular encroachment resulting in severe or fatal crash from Table 66 calculated in the second step.

A worksheet was developed listing the range of percent trucks and encroachments for each hazard environment and the risk associated with each cell was calculated. This was done for each of three risk levels (i.e., 0.005, 0.01 and 0.02). The resulting data was then formulated into tables such as those shown later in Figure 52, Figure 56 and Figure 57. These tables form the basis of the selection process as will be outlined in the next section.

Table 66. Probability a Collision Will Result in a Severe or Fatal Injury by Hazard Environment and MASH Test Level.

	Low Hazard				Medium Hazard				High Hazard			
Percent Trucks	**TL2**	**TL3**	**TL4**	**TL5**	**TL2**	**TL3**	**TL4**	**TL5**	**TL2**	**TL3**	**TL4**	**TL5**
0.00	0.0203	0.0220	0.0016	0.0016	0.0560	0.0571	0.0084	0.0016	0.1251	0.1268	0.0101	0.0016
1.00	0.0207	0.0223	0.0018	0.0017	0.0563	0.0574	0.0086	0.0017	0.1254	0.1270	0.0103	0.0017
2.00	0.0210	0.0226	0.0020	0.0018	0.0567	0.0577	0.0088	0.0018	0.1257	0.1272	0.0105	0.0018
3.00	0.0214	0.0229	0.0022	0.0019	0.0571	0.0580	0.0090	0.0020	0.1260	0.1275	0.0108	0.0020
4.00	0.0218	0.0232	0.0024	0.0020	0.0575	0.0583	0.0093	0.0021	0.1264	0.1278	0.0110	0.0021
5.00	0.0223	0.0236	0.0026	0.0022	0.0579	0.0586	0.0095	0.0022	0.1267	0.1280	0.0112	0.0023
10.00	0.0244	0.0253	0.0038	0.0028	0.0599	0.0602	0.0107	0.0029	0.1285	0.1294	0.0125	0.0030
15.00	0.0268	0.0272	0.0051	0.0035	0.0623	0.0621	0.0121	0.0037	0.1305	0.1309	0.0139	0.0038
20.00	0.0295	0.0293	0.0065	0.0043	0.0648	0.0641	0.0136	0.0045	0.1327	0.1327	0.0154	0.0047
25.00	0.0325	0.0317	0.0081	0.0052	0.0677	0.0664	0.0153	0.0055	0.1352	0.1346	0.0172	0.0057
30.00	0.0358	0.0344	0.0099	0.0062	0.0709	0.0689	0.0172	0.0066	0.1380	0.1367	0.0191	0.0069
35.00	0.0397	0.0374	0.0120	0.0073	0.0746	0.0718	0.0193	0.0078	0.1412	0.1392	0.0214	0.0082
40.00	0.0440	0.0409	0.0143	0.0086	0.0788	0.0751	0.0218	0.0092	0.1448	0.1419	0.0239	0.0097

SELECTION GUIDELINES

The following section presents the recommended selection guidelines and the process for the application of the guidelines for the selection of MASH TL2 though TL5 bridge railings. A risk approach applicable to new construction, rehabilitation and retrofitting is recommended. An alternative cost-benefit approach is provided and discussed following the presentation of the process. A discussion of the process and the policy decisions necessary for implementation are also contained later in this section.

Appendix B includes only the process, without any discussion. Appendix B is presented in a format that could be inserted directly into the AASHTO LRFD Bridge Design Specification or the Roadside Design Guide.

Bridge Rail Risk Assessment Process

The selection of the appropriate MASH test level bridge railing for new or rehabilitation construction is dependent on site-specific conditions and results may differ for each side of the bridge. This process, therefore, should be followed for each bridge edge.

These selection procedures only apply to the bridge railing itself. Providing appropriate guardrail-bridge rail transitions, adequate guardrail approaches, and appropriate terminals and crash cushions are also important considerations in the complete safety performance of the bridge. Users should refer to the AASHTO Roadside Design Guide for guidance on appropriate transitions, approach guardrails and terminals.

The following selection guidelines include six parts: (1) determine the anticipated construction year traffic volume (AADT); (2) estimate the total encroachments expected over the 30-year life of a 1,000-ft section of the bridge; (3) adjust the expected number of encroachments for site-specific conditions; (4) select the test level from the appropriate chart; (5) additional considerations; and (6) if guidelines do not apply. These steps are described in full below.

1. Traffic Conditions – Determine the anticipated construction year traffic volume (AADT) and percent trucks (PT). These selection guidelines assume an annual traffic growth rate of 2% per year and a design life of 30 years. If the anticipated growth rate or design life are significantly different, use the following equation to compute the equivalent construction year traffic volume for use in these selection guidelines:

$$AADT_{EQ} = 0.7430 \cdot AADT_0 \cdot (1+G)^{L/2}$$

where:

$AADT_0$ = The anticipated construction year bi-directional traffic volume (use the one-way traffic volume for one-way roads and ramps),

$AADT_{EQ}$ = The equivalent construction year bi-directional traffic volume,

G = The anticipated annual traffic growth rate where $0 \leq G \leq 1$.

L = The design-life of the bridge railing in years.

2. Encroachments – Estimate the total number of encroachments (N_{ENCR}) that will be experienced on a 1,000-ft section of the bridge railing during the life of the bridge railing by entering Table 68 or Figure 50 with the bi-directional construction year AADT from Step 1 and the highway type.
   a. Do not proportion the value of N_{ENCR} based on the length of the bridge. The entire method is based on a per 1,000-ft basis.
   b. If the AADT of interest falls to the right of the end of the curve for the desired highway type, the level of service for the highway is likely D or worse and these procedures cannot be used; refer to step 6.

3. Site Conditions – Determine the site-specific adjustment factors for the bridge under consideration using the adjustment factors shown in Table 67. Multiply all the adjustments from Table 67 together to obtain f_{TOT}. Find the modified total number of encroachments ($N_{MOD\ ENCR}$) either by:
   a. Drawing a horizontal line in Figure 50 until the curve corresponding to f_{TOT} is obtained (interpolation between lines is acceptable) then reading down to the horizontal axis for the value of the modified total number of encroachments ($N_{MOD\ ENCR}$) on 1,000-ft of bridge railing over the 30-year life of the bridge railing or
   b. Multiplying the estimated encroachments (N_{ENCR}) from Table 68 by the total adjustments (f_{TOT}) from Step 2 to obtain the modified total number of encroachments ($N_{MOD\ ENCR}$) on 1,000-ft of bridge railing over the 30-year life of the bridge railing.

4. Test Level Selection – Characterize the hazard environment under the bridge as *high*, *medium* or *low* according to the following definitions:

HIGH: A high hazard environment below the bridge includes possible interruption to regional transportation facilities (i.e., high-volume highways, transit and commuter rail, etc.) and/or interaction with a densely populated area below the bridge. Penetrating the railing may limit or impose severe limitations on the regional transportation network (i.e., interstates, rail, etc.). Penetrating the railing also has the possibility of causing multiple fatalities and injuries in addition to the injuries associated with the vehicle occupants. A high-hazard environment is also present if penetration or rolling over the bridge railing could lead to the vehicle damaging a critical structural component of the bridge (e.g., a through-truss bridge).

MEDIUM: A medium hazard environment below the bridge includes possible interruption to local transportation facilities, large water bodies used for the shipment of goods or transportation of people, and/or damage to an urban area which is not densely populated. Penetrating the railing would limit local transportation routes, however, detours would be possible and reasonable. Penetrating the railing has the possibility of causing at least one non-motor vehicle injury or fatality.

LOW: A low hazard environment below the bridge includes water bodies not used for transportation, low-volume transportation facilities, or areas without buildings or houses in the vicinity of the bridge. Penetrating a low hazard railing would have little impact on regional or local transportation facilities. A low hazard railing has no buildings or facilities in the area which present possible non-motor vehicle related victims of a rail penetration.

Choose the hazard environment most applicable to the bridge under consideration. Enter the appropriate chart in Figure 52 for the hazard environment selected above, the modified lifetime encroachments per 1,000-ft of bridge edge ($N_{MOD\ ENCR}$) from Step 3, and the percent trucks (PT) from Step 1 to select the appropriate MASH test level for the bridge railing. If the point plots above the dashed risk boundary these charts cannot be used and the engineer should refer to step 6.

5. Additional Considerations – The bridge railing selected using this process provides a solution where the risk of observing a severe or fatal injury crash over the design-life of the bridge railing should be less than 0.01 when the specific site conditions evaluated (i.e., traffic volume and mix, geometry, posted speed limit, and access density) are considered. Engineering judgment should be used when unusual or difficult to characterize site conditions are encountered when selecting a bridge railing. Limited numbers of crash tested bridge railings are available at some test levels, therefore, it is possible that the recommended test level barrier for the evaluated site conditions may not be the best choice for some site conditions not explicitly addressed in these selection guidelines. For example, the particular layout of the barrier at the end of a ramp may influence intersection sight distances and require the use of engineering judgment in designing the interchange to determine an appropriate barrier as it approaches the intersection. Another example might be the presence of pedestrians or bicyclists which might benefit from a higher or different type of railing or the use of sidewalks. Some of the factors that should also be considered are:
    a. TL5 bridge railings may be appropriate for specially designated hazardous material or truck routes.
    b. Intersection sight distance obstructions created by higher test level bridge railings at the ends of ramps or bridges should be considered and the bridge railings may require transitioning to a lower height approaching the intersection.
    c. Stopping sight distance on bridges where the radius and design speed plot below the dashed line in Figure 51 may limit the use of higher test level bridge railings.
    d. The presence of pedestrians, bicyclists, snowmobiles, all-terrain vehicles and other recreational vehicles may affect the choice of bridge railing.

e. Crash history especially as it relates to heavy vehicle crashes or bridge rail penetrations may justify higher performance bridge railings.
f. Regional concerns about snow removal, hydrological impact of flood waters flowing over the bridge, and maintaining scenic views may also play a role in the selection of bridge railings beyond these selection guidelines.
g. The capacity of the bridge deck may limit the choices available for higher test level bridge railings on rehabilitation projects.

6. Guidelines Do Not Apply – There are some situations where these guidelines should not be used, namely:

    a. The traffic conditions violate the free traffic flow assumption used in developing the guidelines such that the estimate of the number of encroachments is not reliable. Generally, this results from a plot point in Figure 50 that is to the right of the end of the highway-type line. This indicates that the level of service may be D or worse and the basic assumptions of the method are invalid.
    b. The user may find that the selection plots above the boundary of Figure 52. In such a case the following options should be considered:
        i. Can the traffic operational conditions (i.e., AADT and percent trucks) be reduced?
        ii. Are the roadway characteristics (e.g., horizontal curvature, grade, etc.) resulting in large adjustments to the N_{ENCR}? Can the geometry be modified to reduce the adjustments?
        iii. Can the deck and superstructure support a TL6 bridge railing?

These situations require a more detailed analysis of the site conditions that examines a broader range of alternatives beyond just the bridge railing test level selection. A solution will probably require the collaboration of traffic operations, geometric design and bridge railing design engineers to either modify the traffic or geometry conditions of the bridge such that these guidelines can be used or perform a crash history investigation to determine the actual performance of the existing bridge railing.

Table 67. Encroachment Adjustments.

Access Density			Lane Width			Horizontal Curve Radius	
Number of Access Points on Bridge or within 200 ft of either end	Undivided	Divided and Oneway	Avg. Lane Width in feet	Undivided	Divided and Oneway	Horizontal Curve Radius at Centerline in feet	All Highway Types
0	1.0	1.0	9≥	1.50	1.25	$950 \geq R_L$	4.00
1	1.5	2.0	10	1.30	1.15	$1910 > R_L > 950$	$(2228.2-R_L)/318.3$
2≤	2.2	4.0	11	1.05	1.03	$1910 \leq R_L$	1.00
			12≤	1.00	1.00	$950 \geq R_R$	2.00
						$1910 > R_R > 950$	$(R_R-2864.8)/954.9$
						$1910 \leq R_R$	1.00
f_{ACC}=			f_{LW}=			f_{HC}=	
Lanes in One Direction			**Posted Speed**			**Grade**	
No. of Through Lanes in One Direction	Undivided	Divided and Oneway	Post Speed Limit	Undivided	Divided and Oneway	Percent Grade	All Highway Types
1	1.00	1.00	<65	1.42	1.18	$-6 \geq G$	2.00
2	0.76	1.00	≥65	1.00	1.00	$-6 < G < -2$	$0.5-G/4$
3≤	0.76	0.91	For roads with unposted speed limits use the adjustment for <65 mi/hr.			$-2 \leq G$	1.00
f_{LN}=			f_{PSL}=			f_G=	

$f_{TOT} = f_{ACC} \cdot f_{LN} \cdot f_{LW} \cdot f_G \cdot f_{HC} \cdot f_{PSL}$ =

Table 68. AADT – Lifetime Encroachments per 1,000-ft of Bridge Railing.

AADT	4 LN DIV	2 LN UNDIV	1 LN ONEWAY	AADT	4 LN DIV	2 LN UNDIV	1 LN ONEWAY
500	0.8	1.2	1.7	33,000	11.6	4.3	
1,000	1.6	1.9	3.4	34,000	11.8	4.4	
2,000	3.1	3.2	6.0	35,000	12.1	4.6	
3,000	4.4	3.6	8.1	36,000	12.4	4.7	
4,000	5.5	3.6	9.6	37,000	12.6	4.8	
5,000	6.5	3.4	10.6	38,000	12.9	5.0	
6,000	7.4	3.1	11.4	39,000	13.2	5.1	
7,000	8.1	2.7	11.8	40,000	13.5	5.2	
8,000	8.8	2.3	12.0	41,000	13.9	5.4	
9,000	9.3	2.0	12.0	42,000	14.2	5.5	
10,000	9.7	1.9	11.9	43,000	14.5	5.6	
11,000	10.1	1.8	11.7	44,000	14.9	5.8	
12,000	10.4	1.8	11.6	45,000	15.2	5.9	
13,000	10.6	1.8		46,000	15.6	6.0	
14,000	10.8	1.9		47,000	15.9		
15,000	10.9	2.0		48,000	16.2		
16,000	11.0	2.1		49,000	16.6		LOS ≥ D
17,000	11.0	2.2		50,000	16.9		
18,000	11.0	2.4		51,000	17.2		
19,000	10.9	2.5		52,000	17.6		
20,000	10.9	2.6		53,000	17.9		
21,000	10.8	2.7		54,000	18.3		
22,000	10.7	2.9		55,000	18.6		
23,000	10.6	3.0	LOS ≥ D	60,000	20.3	LOS ≥ D	
24,000	10.6	3.1		65,000	22.0		
25,000	10.5	3.3		70,000	23.7		
26,000	10.6	3.4		75,000	25.4		
27,000	10.7	3.5		80,000	27.1		
28,000	10.8	3.7		85,000	28.7		
29,000	10.9	3.8		90,000	30.4		
30,000	11.0	3.9		95,000			
31,000	11.2	4.1		100,000	LOS ≥ D		
32,000	11.4	4.2		105,000			
33,000	11.6	4.3		110,000			

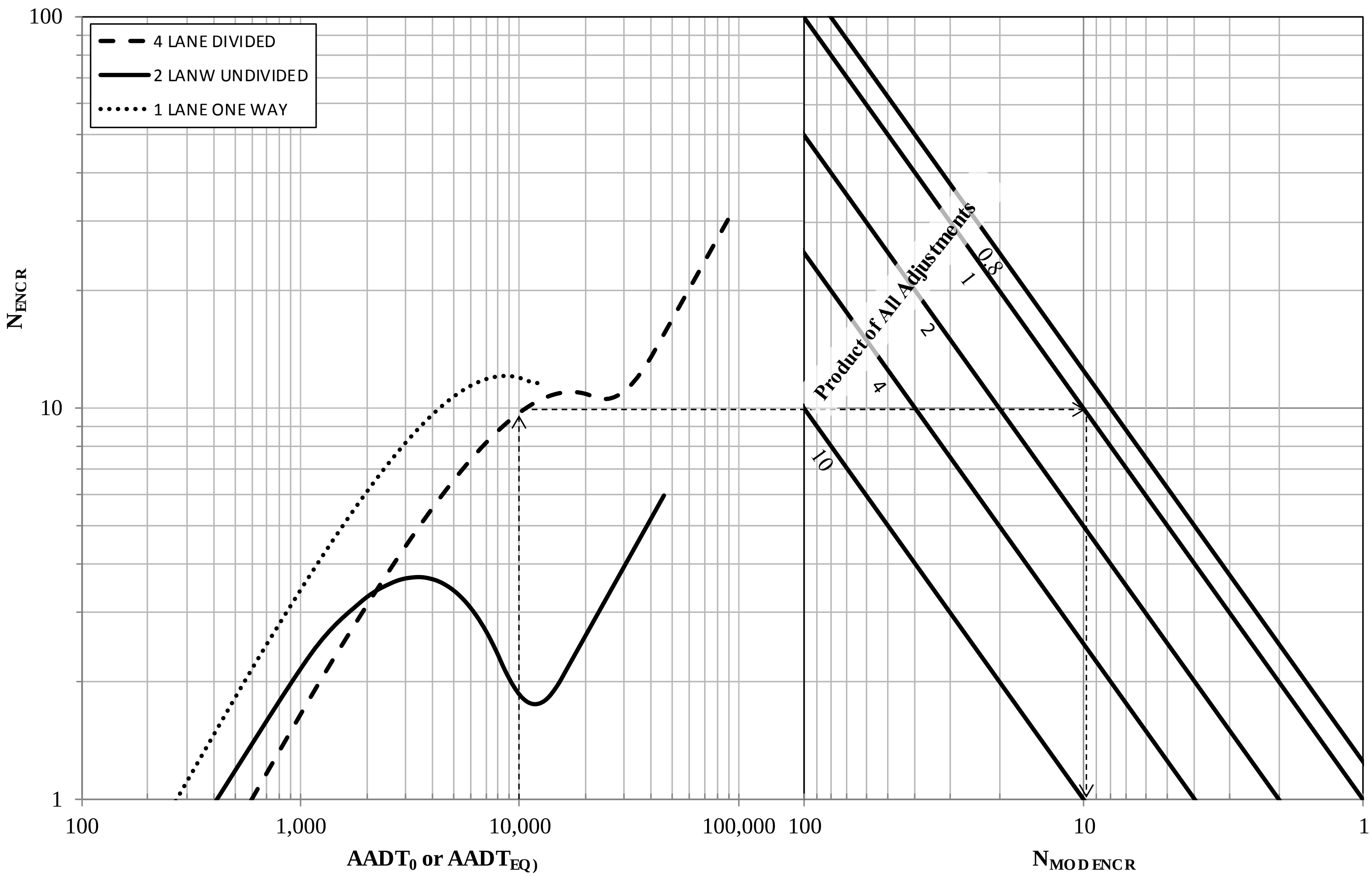


Figure 50. AADT – Lifetime Encroachments/1,000-ft of Bridge Railing Nomograph.

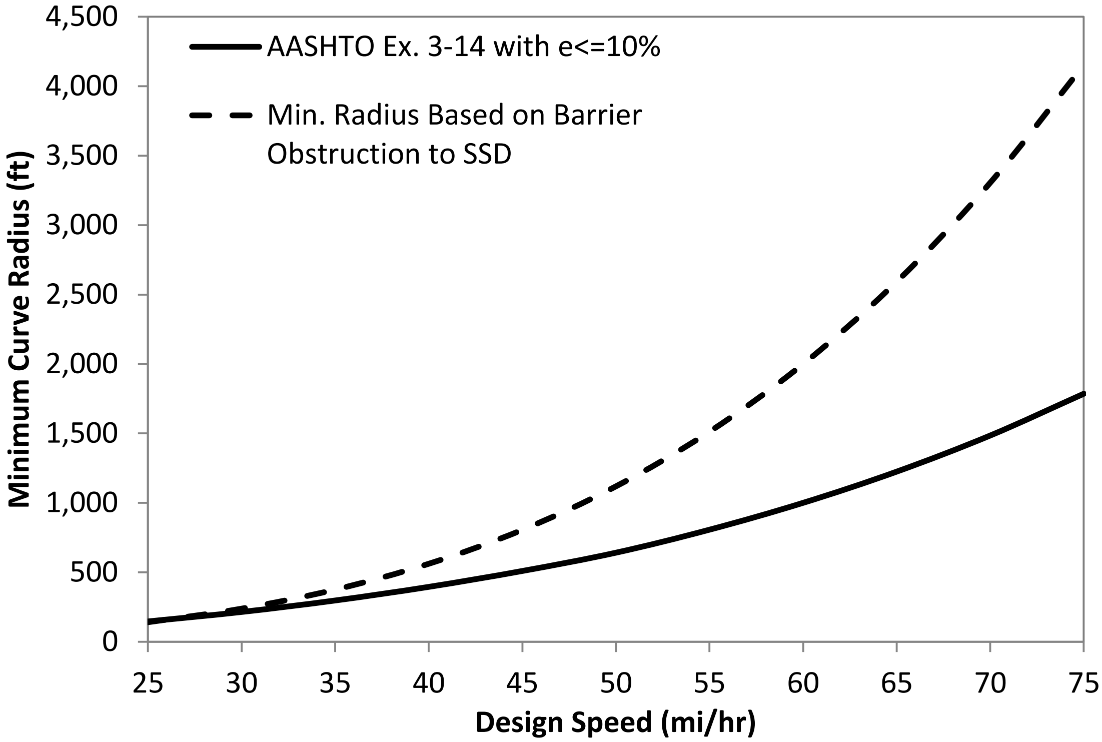
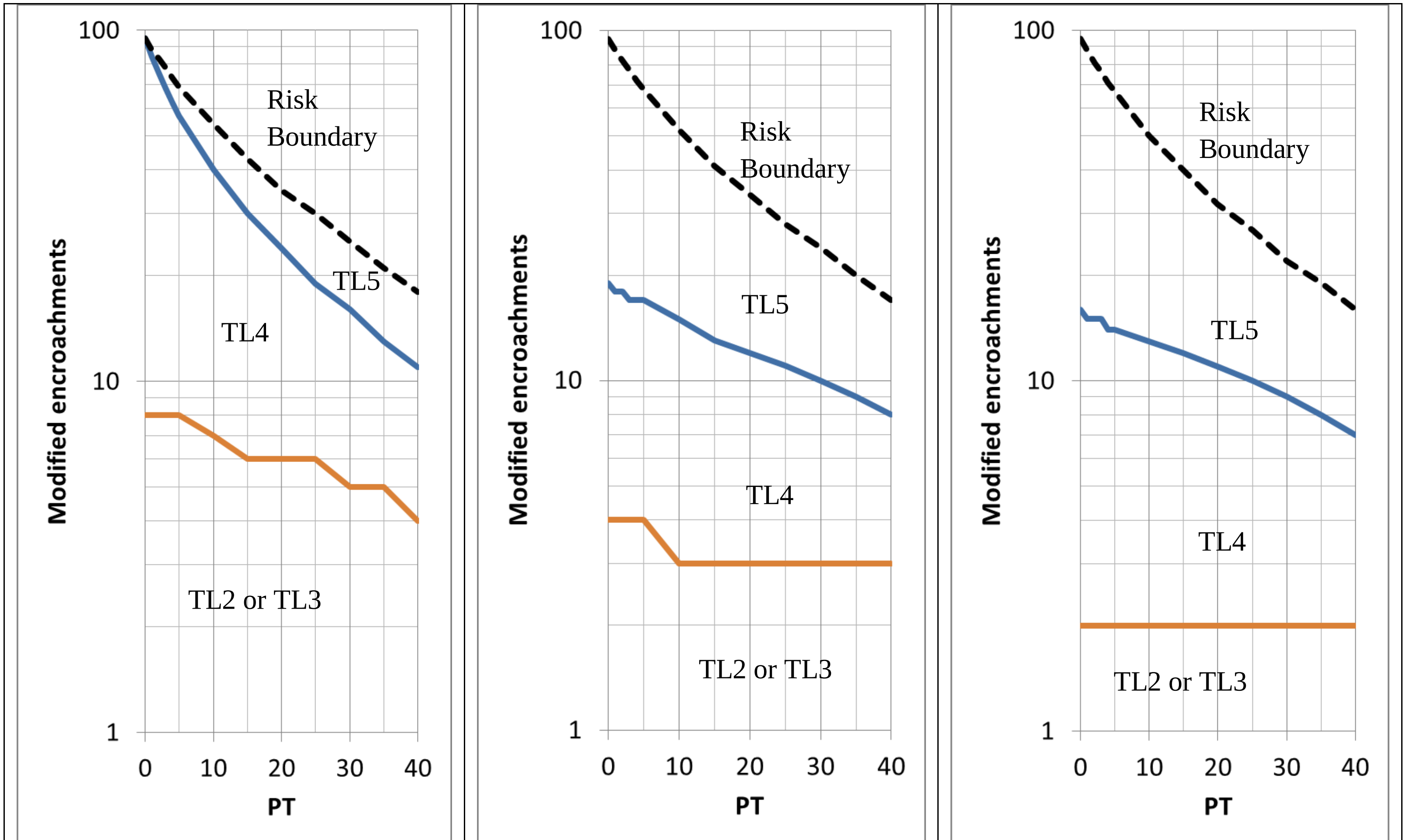


Figure 51. Minimum Horizontal Curve Radius Based on Barrier Obstruction to the Stopping Sight Distance Compared to AASHTO Exhibit 3-14.



LOW	MEDIUM	HIGH

Figure 52. Test Level Selection Nomograph (Risk<0.01 in 30 years for 1000 ft of bridge railing).

Discussion

Implementation

The LRFD Bridge Design Specifications address the issue of the selection of the appropriate test level for bridge railings in Chapter 13 Section 13.7.2 and the Roadside Design Guide addresses the same issue in Chapter 7 Sections 7.3 and 7.5.[AASHTO11, AASHTO12]

Section 13.7.2 of the AASHTO LRFD Bridge Design Specification provides general principles for selecting the appropriate test level for bridge railings. The general principles involve using higher capacity barriers for situations where there are more trucks, higher traffic volumes or particularly hazardous conditions. There is no precise definition of what constitutes a "high" traffic volume or a suitably larger percentage of trucks so it is left to the designer to make a subjective decision regarding how to apply the definitions of Section 13.7.2 to the selection for a particular site.

Chapter 7 of the Roadside Design Guide generally defers to the AASHTO LRFD Bridge Design Specifications with respect to the selection of an appropriate bridge railing. Section 7.3 provides general guidance on bridge railing test level selection and Section 7.5 provides somewhat more detailed descriptions. Section 7.5 provides the following five factors that should be considered in selecting a bridge railing:

1. Performance,
2. Compatibility,
3. Cost,
4. Field Experience and
5. Aesthetics.

Of the five factors, only the first and third (i.e., performance and cost) directly affect the bridge railing selection with respect to the test level. The Roadside Design Guide, in Section 7.5.1, refers to the FHWA policy which requires the use of bridge railings that are crash tested according to Report 350 or subsequent FHWA-approved guidelines such as MASH. [FHWA97a] The FHWA policy also states that "the minimum acceptable bridge railing will be a TL3 … unless supported by a rational selection procedure." [FHWA97a] The selection procedures recommended herein should satisfy the requirement for a "rational selection procedure" for using TL2 bridge railings in particular locations on the NHS with low volumes and low percentages of trucks.

The selection guidelines developed in this project were formulated such that they can be inserted into Section 13.7.2 of the AASHTO LRFD Bridge Design Specification. These new selection guidelines are consistent with the existing wording of Section 13.7.2 and could be inserted after the definitions of the six test levels. It is expected that the AASHTO RDG will continue to defer to the AASHTO LRFD Bridge Specification and refer users to that document although the same guidelines could be inserted into RDG Section 7.5.1 if desired.

Critical Values for Design

The recommended guidelines use a risk-based method where the risk of observing a severe or fatal crash (i.e., A+K) during the design life of the bridge railing was less than 0.01 per 1000-ft of bridge railing. Several policy decisions are inherent in this choice:

- Use of risk rather than benefit-cost,
- A critical value of 0.01 risk of a severe or fatal crash during the 30-year design life of each 1,000-ft of bridge railing,
- 2 percent annual traffic growth and
- The 30-year design life of the bridge railing.

While the recommendations assumed a risk-based method, a critical risk of 0.01 and a design life of 30 years, these values can be changed relatively easily within the context of the procedure outlined above. For example, switching from a risk to a benefit-cost approach is accomplished simply by changing the version of Figure 52; likewise, changing from a risk of 0.01 to 0.02 is accomplished simply by using the appropriate figure in place of Figure 52. Figures that can be used to modify the method type (i.e., risk or benefit cost) and critical value (i.e., various risk or BCR values) are contained in Appendix A. Regardless of which figure is chosen for the final procedures, the process outlined in the last section is exactly the same; only Figure 52 needs to be changed in order to transform the selection procedures from a risk-base to a benefit-cost based procedure or to change the critical design values. If AASHTO SCOBS and TCRS desire to change the recommendations herein, the following replacements can be easily made:

For New Construction

- Risk of 0.005 → Exchange Figure 56 for Figure 52.
- Risk of 0.01 → Use Figure 52 as shown (i.e., the recommended selection guidelines).
- Risk of 0.02 → Exchange Figure 57 for Figure 52.
- Benefit-Cost Ratio of 1 → Exchange Figure 58 for Figure 52.
- Benefit-Cost Ratio of 2 → Exchange Figure 59 for Figure 52.
- Benefit-Cost Ratio of 3 → Exchange Figure 60for Figure 52.

For Rehabilitation Construction

- Risk of 0.005 → Exchange Figure 56 for Figure 52 (i.e., same as for new construction).
- Risk of 0.01 → Use Figure 52 as shown (i.e., the recommended selection guidelines).
- Risk of 0.02 → Exchange Figure 57 for Figure 52(i.e., same as for new construction).
- Benefit-Cost Ratio of 1, 2, or 3
    - Consideration of upgrading from R350 TL4 → Exchange Figure 61 for Figure 52.
    - Consideration of upgrading from R350 TL3→ Exchange Figure 62 and/or Figure 63for Figure 52.

Notice that the selection guidelines are exactly the same for new and rehabilitation construction <u>if</u> a risk-based procedure is used. If a benefit-cost procedure is chosen there will be different versions of the selection figure for new construction versus rehabilitation construction.

The cost-benefit rehabilitation selection figures assume that a Report 350 TL3 or TL4 bridge railing is already in-place and must be demolished and replaced by either a MASH TL4 or TL5 bridge railing.

Test Levels Considerations

Currently, FHWA requires that roadside hardware developed and tested after January 1, 2011 be evaluated according to the AASHTO MASH but still allows the use of hardware designed, tested and accepted under Report 350. [AASHTO09] In developing bridge railing selection guidelines, therefore, there is some ambiguity since new hardware will be evaluated under the MASH criteria but existing hardware tested under Report 350 can and likely will still be used on new or retrofit construction.

Table 69 shows a list of the TL2 through TL5 impact conditions for both the Report 350 and MASH longitudinal barrier crash tests arranged in order of increasing impact severity. One of the difficulties resolved by MASH was that the nominal impact severity of TL3 and TL4 in Report 350 had converged to about 100 ft-kips under Report 350. The MASH TL4 tests were increased in severity, particularly for TL4, in order to provide a broader range of selection options. One of the results is that MASH TL4 barriers generally need to be at least 36-inches tall rather than the 32-inch height that was common for Report 350 TL4.

Table 69. Comparison of Impact Conditions for Report 350 and MASH ordered by Impact Severity.

Test	Vehicle Mass	Speed	Angle	Nominal Impact Severity	Typical Barrier Height
	lbs	mi/hr	deg	ft-kips	in.
R350 TL2	4,409	44	25	50	24
MASH TL2	5,004	44	25	57	Unk
R350 TL4	17,637	50	15	98	32
R350 TL3	4,409	62	25	102	27
MASH TL3	5,004	62	25	116	31
MASH TL4	22,046	56	15	155	36
R350 TL5	79,367	50	15	441	42
MASH TL5	79,367	50	15	441	42

One of the interesting features of developing both the benefit-cost and risk based selection guidelines is that Report 350 TL2 and TL3 and MASH TL3 bridge railings tend to overlap each other. The reason that they overlap is that there is a great deal of performance overlap and relatively little difference in cost. As a result, the TL2 bridge railings generally always appear since the performance difference is small and the cost is slightly less (i.e., even in the risk-based criteria, the least costly railing that meets the risk criteria is the preferred alternative). There are few TL2 bridge railings that were designed and specifically crash tested to the Report 350 TL2 conditions and none at this time to the MASH TL2 conditions. For example, the AASHTO-ARTBA-AGC Guide to Bridge Railings currently contains one concrete and one wood railing that were designed for Report 350 TL2.[TF1313] The construction costs of these new TL2 railings are not well documented since there have only been a handful of installations constructed. Similarly, there is virtually no field crash data experience available so the values used in RSAPv3, while reasonable estimates, cannot be validated with field crash data. There are also a number of older bridge railings that were tested under the AASHTO Guide Specifications for Bridge Railings for PL1 that were "grandfathered" into TL2 when Report 350 was adopted. TL2, as shown in these recommended selection guidelines, can be interpreted as any crash tested bridge railing with a height of less than 27 inches. The recommended selection guidelines offered earlier show TL2 in the recommendations, although SCOBS T-7, TCRS and FHWA may rather change it to MASH TL3 as a policy matter until more data is available on the field performance of these bridge railings.

The Risk Line

Figure 52, as well as its alternate versions in Figure 56 and Figure 57, show a dashed line that indicates the point where even a MASH TL5 bridge railing does not satisfy the selected risk criteria. The line was included so that it is clear to users that the desired criteria could not be met even with a TL5 bridge railing.

It is tempting to consider the risk line a *de facto* indication that a MASH TL6 bridge railing should be used when the point plots above the risk line. In fact, however, a point plotting above the risk line can be interpreted in several different ways including:

- The assumptions built into the development of the process may have been violated,
- There are traffic or roadway conditions that may need to be examined more closely before making a bridge railing selection or
- A TL6 bridge railing may be appropriate.

Knowing which of these situations apply requires some further examination as described in the following sections. Regardless of whether a point plots above the risk line or outside the boundary of the figure chosen for the LRFD, Step 6 in the selection guidelines was added to provide some guidance to engineers who encounter situations where the guidelines may not be appropriate for use.

The selection figures were developed using RSAPv3 and the encroachment model in RSAPv3 assumes that traffic is generally in a free-flowing condition. This has been interpreted to mean that the level of service is at least C or better. Of course sites with poor levels of service do not operate at those levels at all hours. A particular highway might have a level of service of D or F in peak hours but operate at B or even A conditions at the off-peak hours. At this time it simply not known what the effect of degraded levels of service are on the encroachment models so the predictions from RSAPv3 may not be reliable. Further research is needed to determine how the encroachment relationships change at high traffic volumes and levels of service of D or E.

If the traffic volumes for a particular highway type result in a level of service of D or greater, the resulting selection may plot above the risk boundary. To prevent this from happening, Figure 50 and Table 68 were constructed such that each line representing a highway type ends at the AADT corresponding to the transition from level of service C to D for 40 percent trucks. The engineer is also told explicitly in step 2b not to extrapolate to the right of the highway-type lines because doing so will violate the basic assumptions used to develop the tables. If a particular AADT plots to the right of the highway-type line, the user is directed to step 6 for advice. A selection above the risk boundary should not be the result of poor level of service as long as the user has followed the instructions in step 2.

More important is the fact that the risk line indicates that there are probably other issues related to the site or traffic conditions which a simple choice of bridge railing test level may not adequately address. The selection figures use the expected number of life-time encroachments on a 1,000-ft section, the percent of trucks and the hazard environment as input to select a bridge railing. If a particular bridge situation plots above the risk line or outside the boundary of the figure then the combination of these three input values have resulted in a situation where one of the other inputs should be considered for change.

The hazard environment determines which figure is used in the selection process. The hazard environment is determined based on the character of the area beneath and around the bridge. Changing the hazard environment is generally not a feasible alternative since it involves land use outside the typical DOT’s right of way and control or very expensive changes to the transportation infrastructure. For example, a gasoline tank farm may be located beneath a bridge causing the hazard environment to be categorized as a high. While moving the tank farm is a theoretical possibility it is probably not practical due to the expense involved as well as the need to coordinate and collaborate with private property owners and a variety of local agencies. For these types of reasons, changing the hazard environment is generally not an option.

Sometimes selections below the risk boundary could be made if the percent trucks were reduced thereby moving the plot point to the left. If a particular bridge plots above the hazard line the engineer may want to consider if the high percentage of truck traffic is desirable at the site and if there are ways to reduce the truck traffic in the long term. Returning to the tank farm example, if the site experienced 30 percent trucks and was located in a heavily urban area it might make sense to consider redirecting truck through-traffic to another route to avoid the high hazard areas (e.g., loop route around urban area). Reducing the percent trucks from 30 to 10 may be enough to reduce the risk below the risk line. Obviously, this alternative has consequences far afield from the consideration of the one bridge under consideration since it would involve a change in the operation of the highway network and how traffic is managed.

Probably the most important consideration, however, is to examine why so many encroachments are predicted at the site. There are several possible reasons for predicting a large number of encroachments including (1) very high traffic volumes and/or (2) geometric characteristics that result in large adjustments. The adjustment factors can increase the expected number of encroachments dramatically. For example, if a one-way ramp has a radius of horizontal curvature of 950 ft, a six percent downhill grade and a speed limit of 45 mi/hr, Table 67 indicates an encroachment adjustment of 9.44 is needed. This is a very high adjustment indicating that the geometry of the roadway may be very challenging. This adjustment applied to encroachments for a moderate AADT could well place the site above the risk boundary. The engineer should seriously consider addressing the curvature and grade of the site to reduce the number of encroachments expected and bring the site conditions below the risk boundary.

While a MASH TL6 bridge railing might be appropriate for some conditions above the dotted risk line, there is at present only one crash tested TL6 bridge railing, the 90-inch tall TX T80TT bridge railing. [TXDOT13] This bridge railing requires the use of non-standard deck details since the dead-load of the bridge railing and overturning resistance are so large. Because it cannot be used on a conventional deck it is not generally a viable option for most bridge construction projects. Before selecting a TL6 bridge railing, the engineer will need to carefully examine the structural characteristics of the deck and bridge structure as well as traffic and site conditions to determine if a TL6 bridge railing is a realistic alternative. There is no guaranteed that even if a TL6 bridge railing is used it will satisfy the risk criteria since sometimes the added risk is a result of very high traffic volumes and passenger vehicle redirection-rollovers which are likely not improved with a TL6 bridge railing.

As these examples illustrate, when a point plots above the risk boundary in Figure 52 reducing the risk will almost always involve a more comprehensive approach than simply selecting a bridge railing. Reducing the risk may involve highway geometric design, traffic operations and management, structural design of the bridge deck and superstructure as well as the selection of a bridge railing.

Recommended Selection Guidelines Verification

The 1989 AASHTO Guide Specification to Bridge Rails, NCHRP Report 22-08 and a series of example problems were reviewed and compared to the recommended procedure as a verification exercise.

Comparisons to the 1989 AASHTO Guide Specification and NCHRP 22-08

The 1989 AASHTO Guide Specification for Bridge Railings and the selection guidelines that were prepared in NCHRP 22-08 have been converted to the same format as the proposed selection guidelines for comparison purposes. These converted tables are shown in Figure 53 on the left side. While the purpose of this project is not to mimic the 1989 AASHTO GSBR guidelines, those earlier guidelines do provide some insight into what roadside safety engineers in the past have considered “reasonable” selection guidelines. The proposed selection guidelines from this project are compared to the 1989 GSBR and the NCHRP 22-08 selection guidelines to gauge how these new guidelines compare to what was accepted to some degree in the past.

Figure 53 shows the recommended risk figure for a medium hazard in the center overlaid on the 1989 GSBR guidelines. It appears the recommendations from this research are slightly more conservative than the 1989 GSBR for a lifetime risk of 0.01. Figure 53 also shows the risk of 0.03, which more closely matches the 1989 GSBR guidelines. Figure 54 provides a comparison of the GSBR with the recommended BCR approach. Figure 55 directly compares the GSBR selection guidelines, the recommended risk-based approach and the corresponding cost-benefit approach.

Each of these tables have underlying assumptions and adjustments which cannot be captured in a straight-forward comparison of the charts. Furthermore, the GSBR was developed using bridge rails designed to a different performance specification. While no official crash test equivalencies have been released to compare Report 350 and MASH test level, Table 70 was originally released by the FHWA to compare the GSBR performance levels with Report 350 and was expanded to add the first line to represent the approximately equivalencies for MASH test levels.

Table 70. Approximate Crash Test Acceptance Equivalencies. [after Horne97]

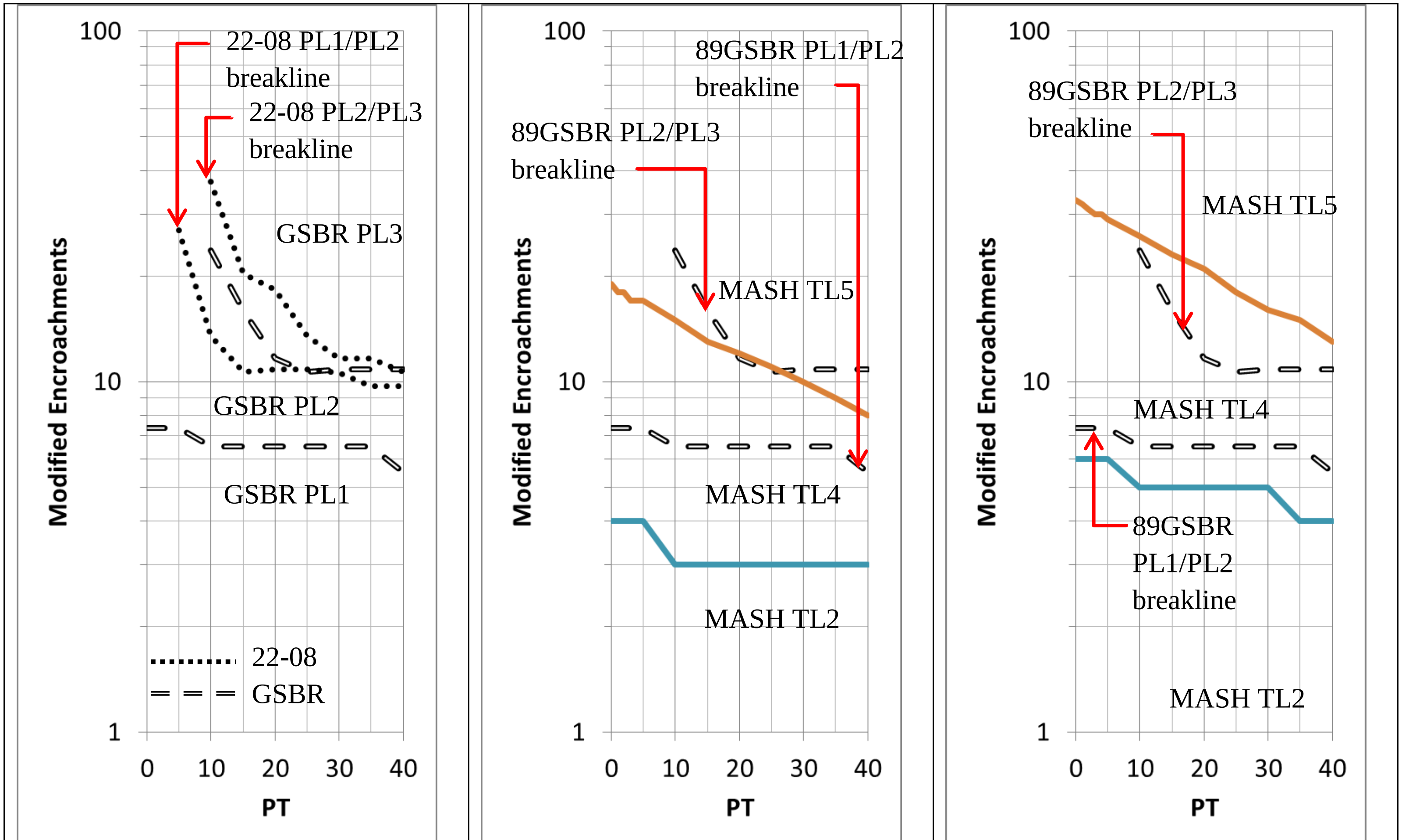
Bridge Railing Testing Criteria		Acceptance Equivalencies					
MASH	TL1	TL2		TL3	TL4	TL5	TL6
Report 350	TL1	TL2	TL3	TL4		TL5	TL6
Report 230		MSL-1 MSL-2†					
1989 AASHTO Guide Spec		PL1		PL2		PL3	
AASHTO LRFD Bridge Spec		PL1		PL2		PL3	

† 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.

Each of these guidelines has been used to evaluate a series of example problems where the bridge rail is already in place as a verification exercise.

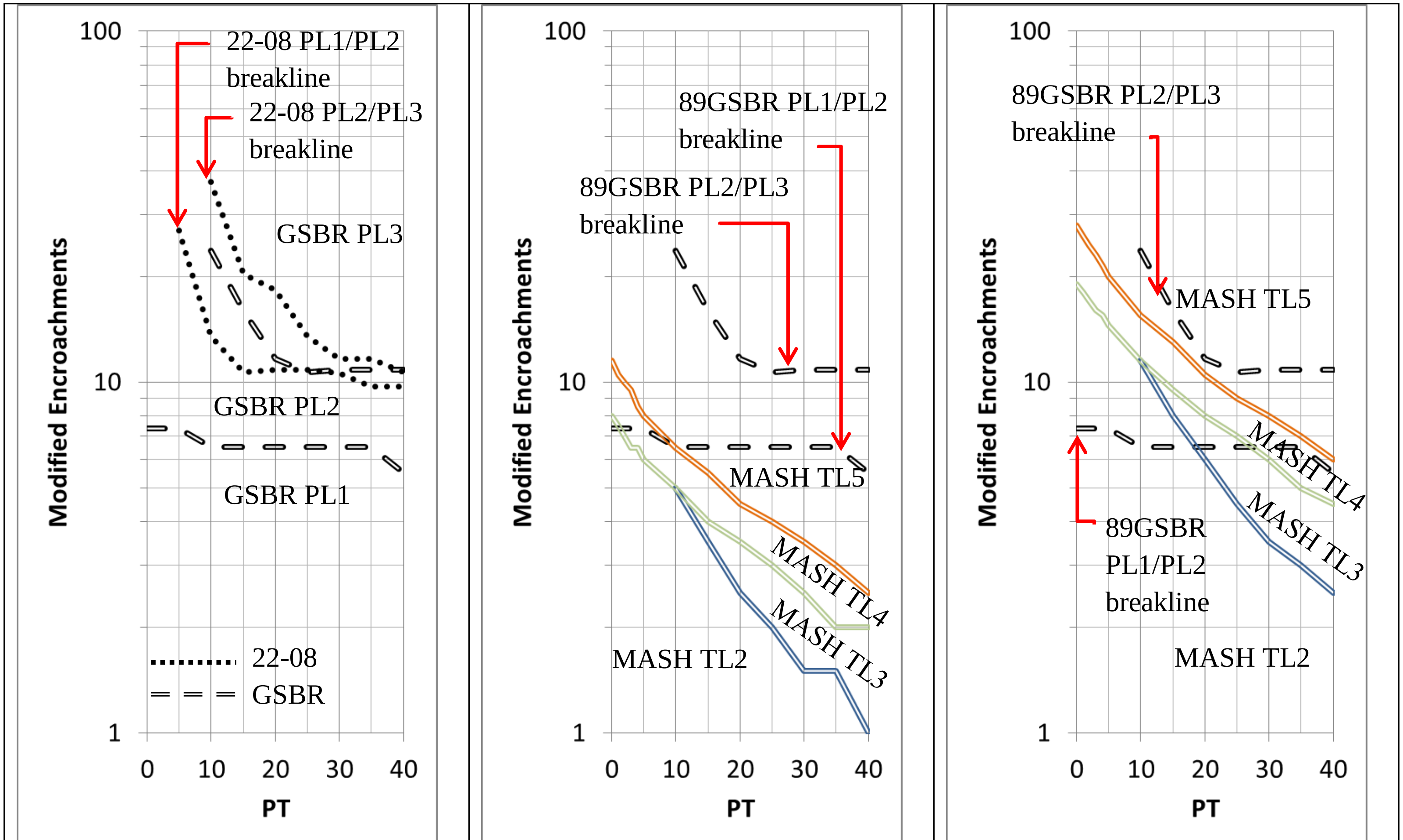
Example Bridge Railing In Service

Table 71 shows several example bridge railings along with the characteristics of the site, traffic conditions and the type of bridge railings currently installed at the bridge. These examples are meant to compare what some agencies currently have installed and compare the current installation to the bridge railings selected by the recommended procedure and the 1989 GSBR. Table 71 also serves as a verification exercise since the bridge railing selection determined from the recommended procedure is compared to the result recommended by an independent RSAPv3 analysis. While the recommended selection guidelines are based on numerous RSAPv3 simulations, the results were re-formulated, rearranged and simplified to develop the recommended procedure so it was important to verify that the final procedure was consistent with individually generated results from RSAPv3. As shown in Table 71, the recommended selection guidelines produce the same or slightly more conservative results than performing an RSAPv3 analysis. This verification exercise was completed for each risk level and benefit-cost ratio presented in the alternate figures shown in Appendix A.



1989 GSBR and 22-08 Findings	Risk≤0.01 overlaid on 89GSBR	Risk≤.03 overlaid on 89GSBR

Figure 53. Comparison of 89GSBR, NCHRP22-08 with Risk values.



1989 GSBR and 22-08 Findings	BCR=2 overlaid on 89GSBR	BCR=5 overlaid on 89GSBR

Figure 54. 89GSBR, NCHRP22-08 with BCR values.

<table>
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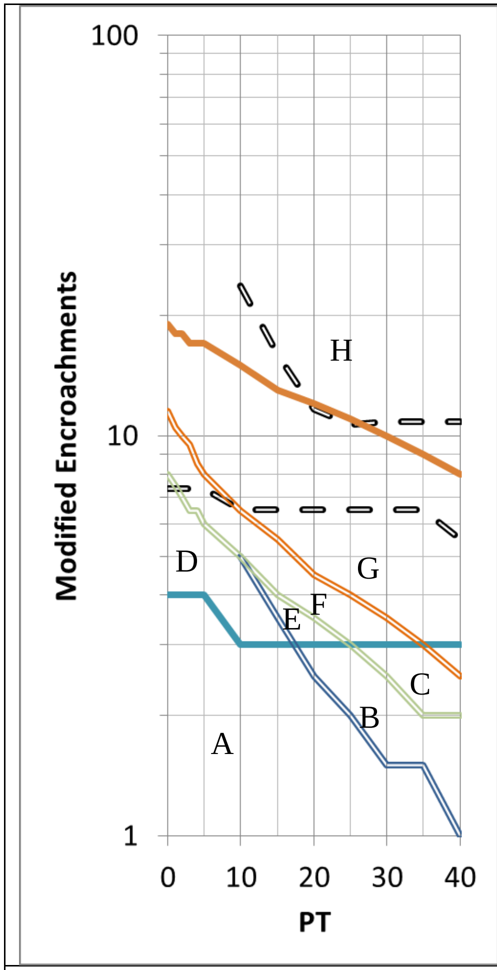
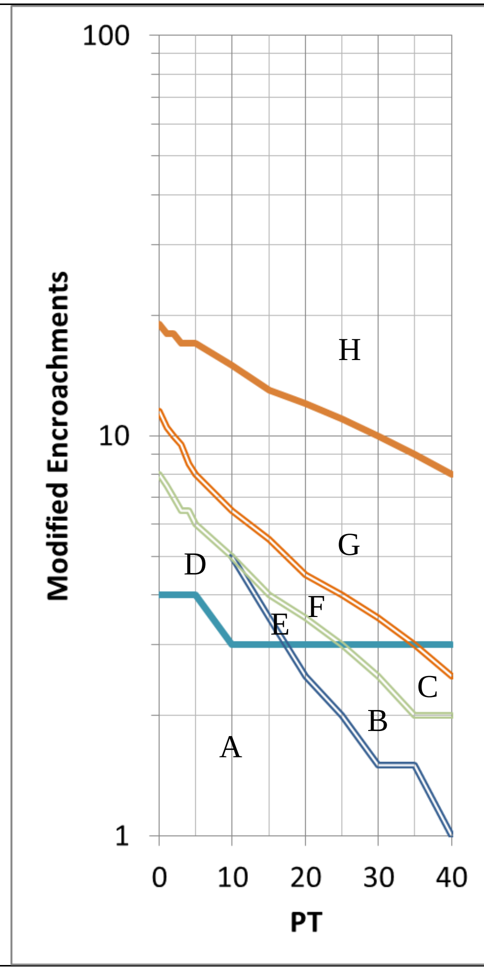
<td>
</td>
<td>
</td>
<td>A. MASH TL2 satisfies risk and cost-benefit criteria.
B. MASH TL2 satisfies risk criteria, MASH TL3 is cost-beneficial.
C. MASH TL2 satisfies risk criteria, MASH TL4 is cost-beneficial.
D. MASH TL4 satisfies risk criteria, MASH TL2 is cost-beneficial.
E. MASH TL4 satisfies risk criteria, MASH TL3 is cost-beneficial.
F. MASH TL4 satisfies risk criteria, MASH TL4 is cost-beneficial.
G. MASH TL4 satisfies the risk criteria, MASH TL5 is cost-beneficial.
H. MASH TL5 satisfies the risk criteria, MASH TL5 is cost-beneficial.

A-F comparable to 89GSBR PL1
G comparable to 89 GSBR PL2
H is comparable to 89 GSBR PL3</td>
</tr>
<tr>
<td>1989 GSBR overlaid on findings</td>
<td>Risk≤0.01 and BCR=2</td>
<td>Comparison of Risk and CBR tables</td>
</tr>
</table>

Figure 55. 89GSBR, Risk and BCR comparison.

Table 71. Selected Examples of Existing Bridge Railings Compared to the Recommended Selection Guidelines.

UNDIVIDED HIGHWAYS				
Location			**Characteristics**	**Photograph**
ME SR140 over the Androscoggin River in Canton ME			ADT =1,710 vpd PT =22% Haz. Env. = Low Grade = +/-4% Curv. Rad. = Tangent Acc. Pts. = 0 Shldr = 4 ft No. Prim. Lns. = 1 PSL = 50 mi/hr f_{TOT} = 2.13 N_{ECNR} = 3.1	
Currently Installed	R350 TL4 2-bar NETC			
1989 AASHTO GSBR	PL1			
	RSAP	**Procedure**		
Risk<0.005	MASH TL4	MASH TL4		
Risk<0.010	MASH TL4	MASH TL4		
Risk<0.020	MASH TL4	MASH TL2/3		
BCR=1	MASH TL3	MASH TL5		
BCR=2	MASH TL3	MASH TL3		
BCR=3	MASH TL3	MASH TL3		
Location			**Characteristics**	**Photograph**
Rt 15 over the Piscataquis River in Guilford ME. There are intersections on each end of the bridge			ADT =3,780 vpd PT = 15% Haz. Env. = Low Grade = Flat Curv. Rad. = Tangent Acc. Pts. = 2 Shldr = 7 ft No. Prim. Lns. = 1 PSL = 55 mi/hr Length = 170 ft f_{TOT} = 3.12 N_{ECNR} = 3.6	
Currently Installed	Obsolete			
1989 AASHTO GSBR	PL2			
	RSAP	**Procedure**		
Risk<0.005	MASH TL4	MASH TL4		
Risk<0.010	MASH TL4	MASH TL4		
Risk<0.020	MASH TL4	MASH TL4		
BCR=1	MASH TL4	MASH TL5		
BCR=2	MASH TL4	MASH TL4		
BCR=3	MASH TL4	MASH TL3		

Table 71. Selected Examples of Existing Bridge Railings Compared to the Recommended Selection Guidelines. (continued)

UNDIVIDED HIGHWAYS				
Location			**Characteristics**	**Photograph**
SR 20 over the French River in Oxford MA			ADT =25,428 vpd	
			PT = 5%	
Currently Installed	Unknown		Haz. Env. = Low	
1989 AASHTO GSBR	PL2		Grade = Flat	
	RSAP	**Procedure**	Curv. Rad. = Tangent	
Risk<0.005	MASH TL2	MASH TL2	Acc. Pts. = 0	
Risk<0.010	MASH TL2	MASH TL2	Shldr = 2 ft	
Risk<0.020	MASH TL2	MASH TL2	Sidewalk = 5 ft	
BCR=1	MASH TL2	MASH TL3	No. Prim. Lns. = 2	
BCR=2	MASH TL2	MASH TL3	PSL = 50 mi/hr	
BCR=3	MASH TL2	MASH TL3	f_{TOT} = 1.08	
			N_{ECNR} = 3.3	
Location			**Characteristics**	**Photograph**
SR 122A over SR 146, a railroad yard and bikepath in Worcester MA			ADT =11,555 vpd	
			PT = 8%	
Currently Installed	R350 TL4 Vertical Wall		Haz. Env. = High	
1989 AASHTO GSBR	PL2		Grade = Flat	
	RSAP	**Procedure**	Curv. Rad. = Tangent	
Risk<0.005	MASH TL4	MASH TL4	Acc. Pts. = 2	
Risk<0.010	MASH TL4	MASH TL4	Shldr = 2 ft	
Risk<0.020	MASH TL4	MASH TL4	Sidewalk = 5 ft	
BCR=1	MASH TL4	MASH TL5	No. Prim. Lns. = 2	
BCR=2	MASH TL3	MASH TL4	PSL = 50 mi/hr	
BCR=3	MASH TL2	MASH TL3	f_{TOT} = 2.3	
			N_{ECNR} = 1.8	

Table 71. Selected Examples of Existing Bridge Railings Compared to the Recommended Selection Guidelines. (continued)

DIVIDED HIGHWAYS				
Location			**Characteristics**	**Photograph**
I-190 over Princeton Rd in Sterling MA			ADT =48,500 vpd	
			PT =10%	
Currently Installed	R350 TL4 F Shape		Haz. Env. = Low	
1989 AASHTO GSBR	PL2		Grade = Flat	
	RSAP	**Procedure**	Curv. Rad. = Tangent	
Risk<0.005	MASH TL4	MASH TL4	Acc. Den. = 0 pts/mi	
Risk<0.010	MASH TL4	MASH TL4	Rt. Shldr = 12 ft	
Risk<0.020	MASH TL4	MASH TL4	Lt. Shldr = 2 ft	
BCR=1	MASH TL4	MASH TL5	No. Prim. Lns. = 2	
BCR=2	MASH TL4	MASH TL4	Med. Width = 190 ft	
BCR=3	MASH TL4	MASH TL4	fTOT = 1.00	
			N_{ECNR} = 16.4	
Location			**Characteristics**	**Photograph**
New Jersey Turnpike over SR 30 in NJ			ADT =59,450 vpd	
			PT =15%	
Currently Installed	R350 TL4 NJ w/ top rail		Haz. Env. = Medium	
1989 AASHTO GSBR	PL3		Grade = Flat	
	RSAP	**Procedure**	Curv. Rad. = Tangent	
Risk<0.005	MASH TL5	MASH TL5	Acc. Den. = 0 pts/mi	
Risk<0.010	MASH TL5	MASH TL5	Rt. Shldr = 10 ft	
Risk<0.020	MASH TL5	MASH TL5	Lt. Shldr = 12 ft	
BCR=1	MASH TL5	MASH TL5	No. Prim. Lns. = 2	
BCR=2	MASH TL5	MASH TL5	Med. Width = 27 ft	
BCR=3	MASH TL5	MASH TL5	f_{TOT} = 1.00	
			N_{ECNR} = 20.3	

Table 71. Selected Examples of Existing Bridge Railings Compared to the Recommended Selection Guidelines. (continued)

ONE WAY HIGHWAYS				
Location			**Characteristics**	**Photograph**
I-290 Exit 15 Ramp over Shrewsbury Street, Worcester MA			ADT =6,280 vpd PT =2%	
Currently Installed	R350 TL4 NJ		Haz. Env. = High	
1989 AASHTO GSBR	PL2		Grade = -4%	
Risk<0.005	MASH TL5	MASH TL5	Curv. Rad. = 300 ft	
Risk<0.010	MASH TL5	MASH TL5	Acc. Den. = 0 pts/mi	
Risk<0.020	MASH TL5	MASH TL4	Rt. Shldr = 2 ft	
BCR=1	MASH TL4	MASH TL5	Lt. Shldr = 2 ft	
BCR=2	MASH TL4	MASH TL5	No. Prim. Lns. = 1	
BCR=3	MASH TL3	MASH TL5	f_{TOT} = 7.08	
			N_{ECNR} = 10.2	

CONCLUSIONS

The objective of this project was to develop recommended guidelines for the selection of Test Levels 2 through 5 bridge rails considering in-service performance, site and traffic conditions such as traffic volume, percent trucks and land use under and around the bridge. Currently, only general guidance is available from FHWA and AASHTO on when to use one of the six test levels identified in MASH. State DOT policy makers, bridge engineers and highway designers have little basis for their decisions aside from the general philosophical guidance contained in Chapter 7 of the Roadside Design Guide and Chapter 13 of the AASHTO LRFD Bridge Design Specifications. This project filled that gap using a risk-based approach based on risk and cost-benefit analyses performed with version 3.0.1 of the Roadside Safety Analysis Program (RSAPv3). The results of this project were selection guidelines for choosing the appropriate test level for bridge railings based on site and traffic conditions as well as the observed crash performance of common bridge railings. These selection guidelines, as presented earlier, will be useful to designers and State DOTs for selecting the appropriate bridge railings based on particular site conditions.

The resulting selection guidelines are based on the risk of observing a severe or fatal crash during the 30-year life of the bridge railing less than or equal to 0.01 for a 1000-ft long bridge railing. The guidelines explicitly use the traffic volume and percent of trucks as well as the geometric characteristics of the bridge like the horizontal curvature, grade and number of lanes to estimate the likely number of vehicles that will leave the travelled way and strike the bridge rail. The crash performance characteristics of modern crash tested bridge railings were included by examining and modeling real-world crash data to predict the occupant injury, post-impact trajectory and probability of penetrating, rolling over or vaulting over the bridge railing. The area around and surrounding the bridge has been characterized into three categories: low hazard, medium hazard and high hazard. Low hazard areas are those where penetrating the bridge railing places only the occupants of the impacting vehicle at risk whereas high hazard areas have the potential for catastrophic loss of life. The selection guidelines use these characteristics of the bridge, traffic and surrounding area as input and present a recommendation for the appropriate MASH test level bridge railing that will result in a risk equal to or less than 0.01 of observing a severe or fatal crashes over the 30-year life of 1,000-ft of the bridge. These selection guidelines, therefore, make choosing the appropriate bridge railing a function of the characteristics of the bridge and the potential for catastrophic harm if the bridge railing is penetrated.

In developing these selection guidelines several areas for improvement were identified that would require further research. First, little is known about the nature and character of vehicle encroachment at high traffic volumes, especially as service level conditions degrade from the free-flow. A better understanding of encroachment rates at poor service level conditions and high traffic volumes would help make these guidelines applicable to a broader range of traffic conditions, especially on heavily travelled urban corridors. Second, there is relatively little known about the encroachment characteristics of heavy vehicles. This project used what little data is available to estimate the encroachment rates and trajectories of heavy vehicles but there is a need for a more comprehensive approach to predicting the frequency and extent of heavy vehicle encroachments. This research was focused exclusively on selecting an appropriate bridge railing but it is also important to ensure that the guardrail-bridge rail transitions, approach guardrail and terminals are also adequately designed. Ultimately, the designer is responsible for the complete safety system on the bridge not just the selection of the bridge railing test level so further research aimed at better specifying how these test level selections should be integrated with the transitions and approach guardrails is a logical next step. While additional research in these areas would help to extend the applicability of these selection guidelines, the guidelines presented herein are a good first step since they should adequately account for the majority of bridge railing selection situations.

APPENDIX A: ALTERNATIVE GRAPHS FOR FIGURE 52

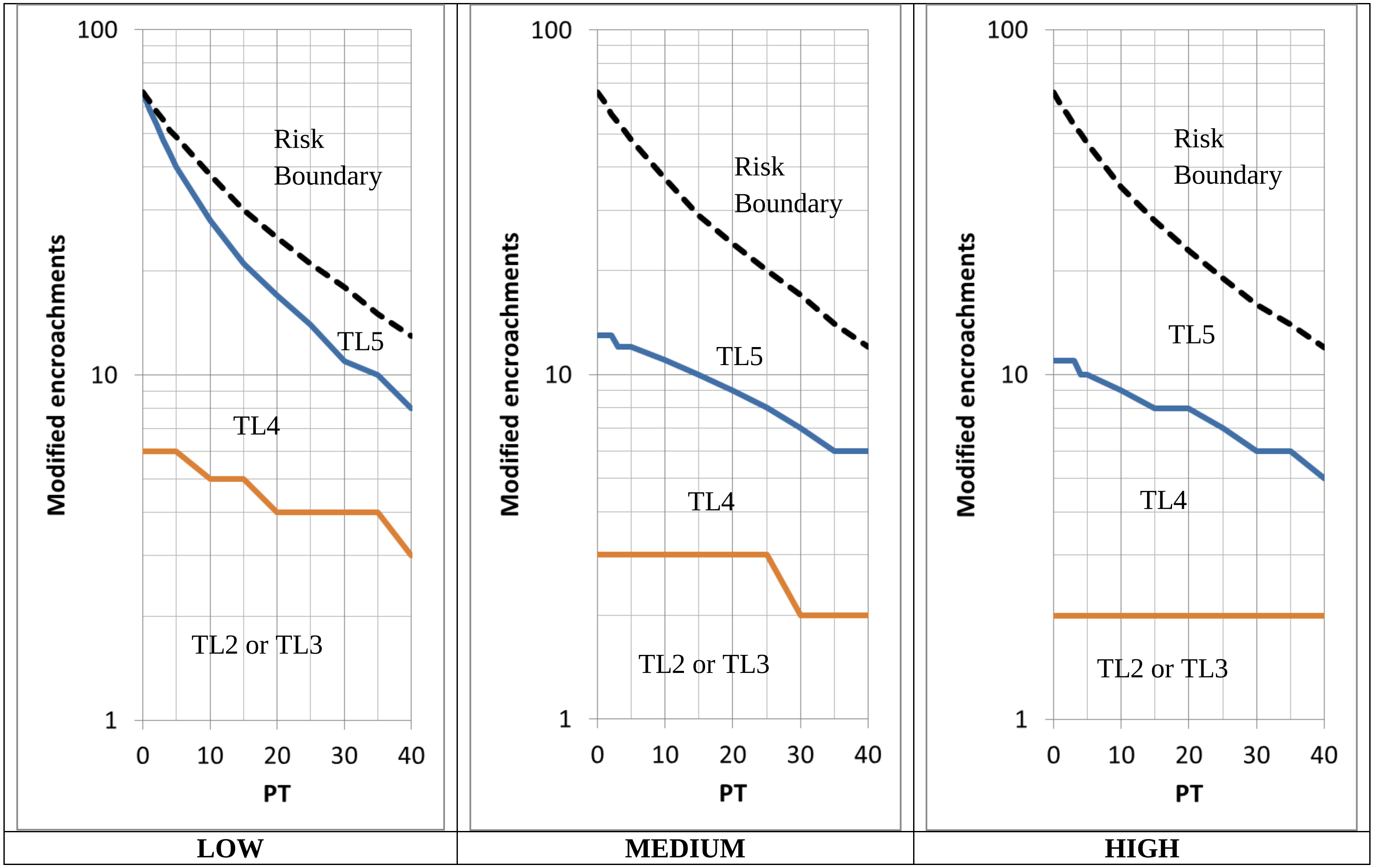
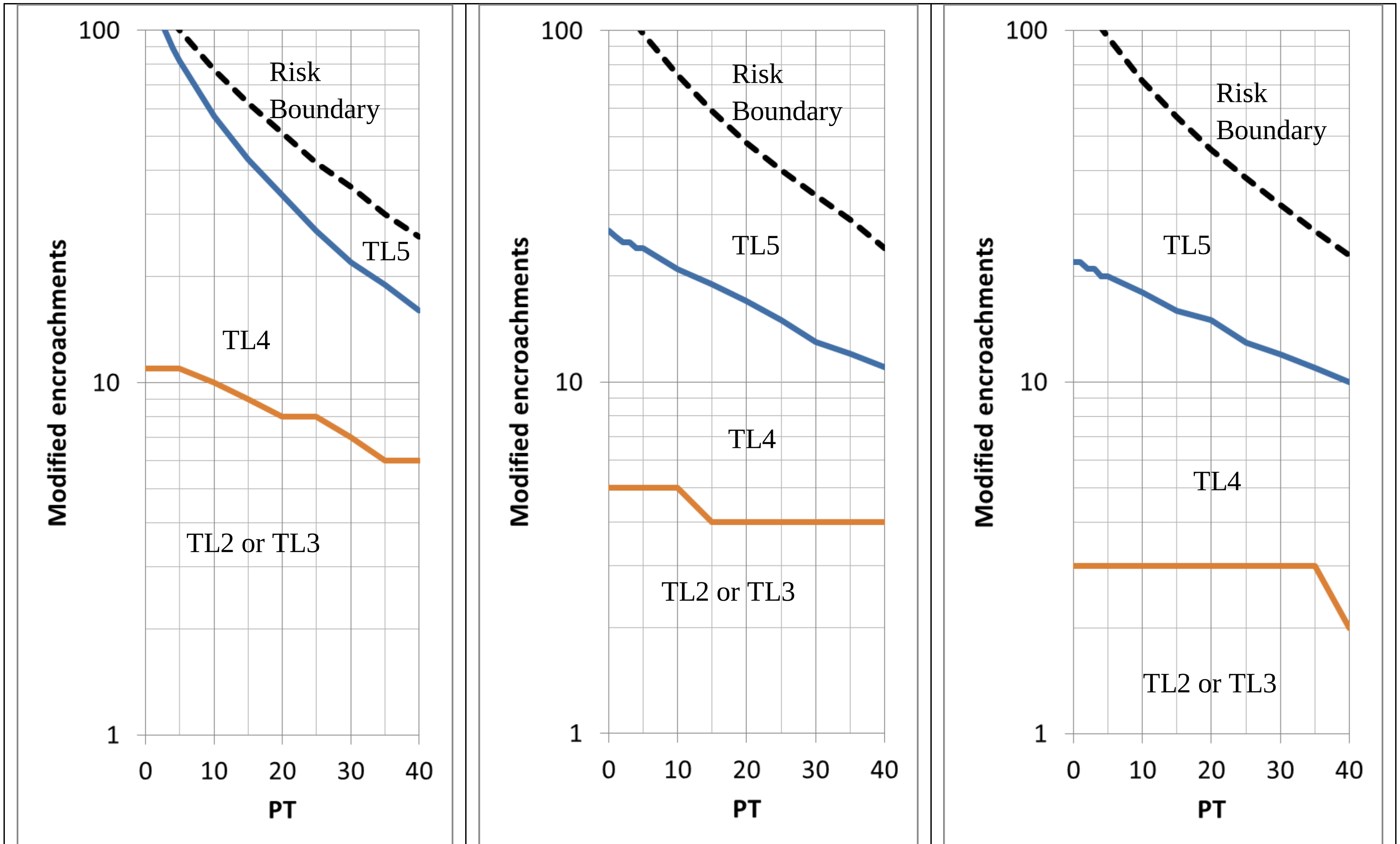
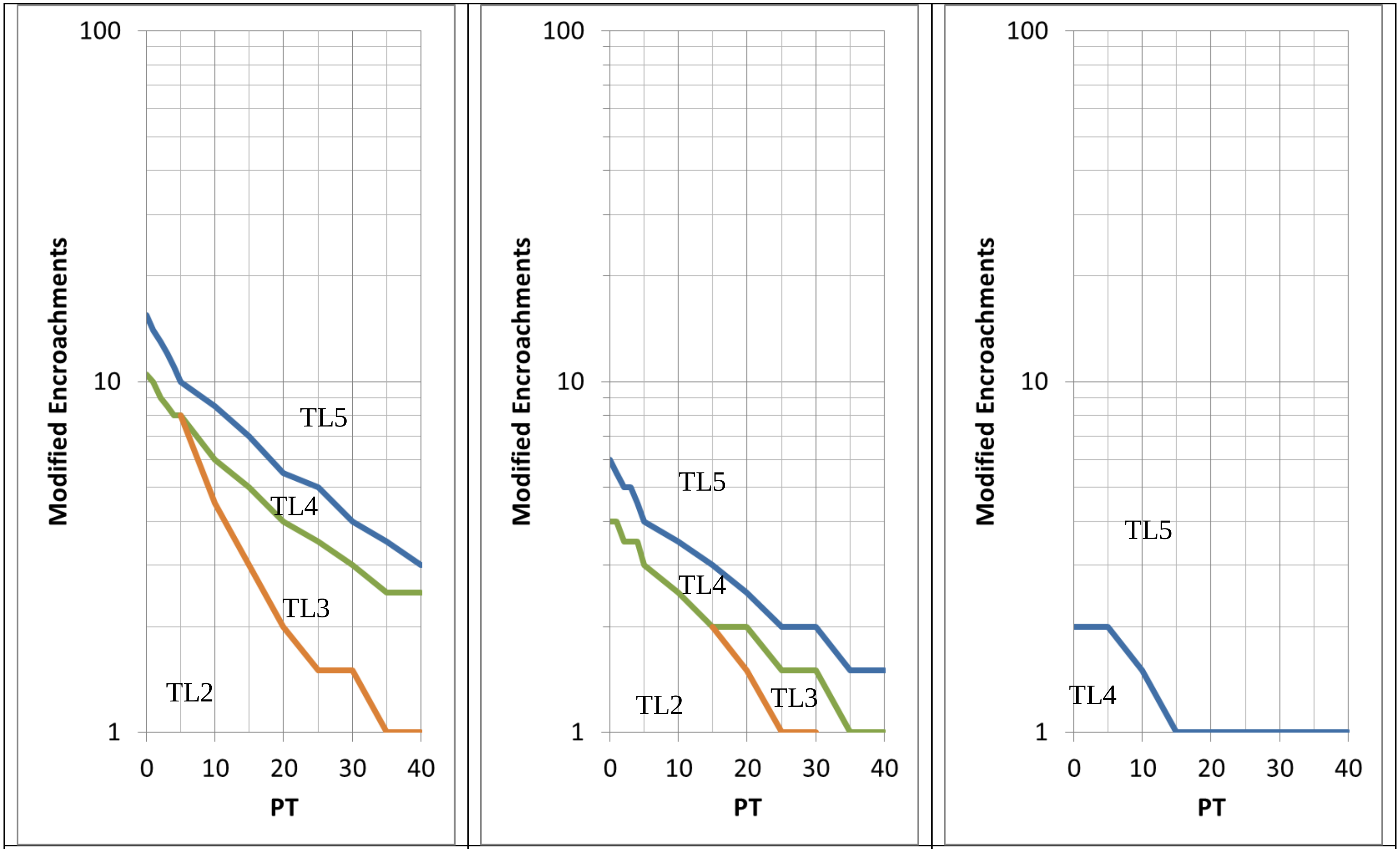


Figure 56. Test Level Selection Nomograph (Risk<0.005 in 30 years for 1000 ft of bridge railing).



LOW	MEDIUM	HIGH

Figure 57. Test Level Selection Nomograph (Risk<0.02 in 30 years for 1000 ft of bridge railing).



LOW	MEDIUM	HIGH

Figure 58. Test Level Selection Nomograph (Benefit-Cost Ratio=1).

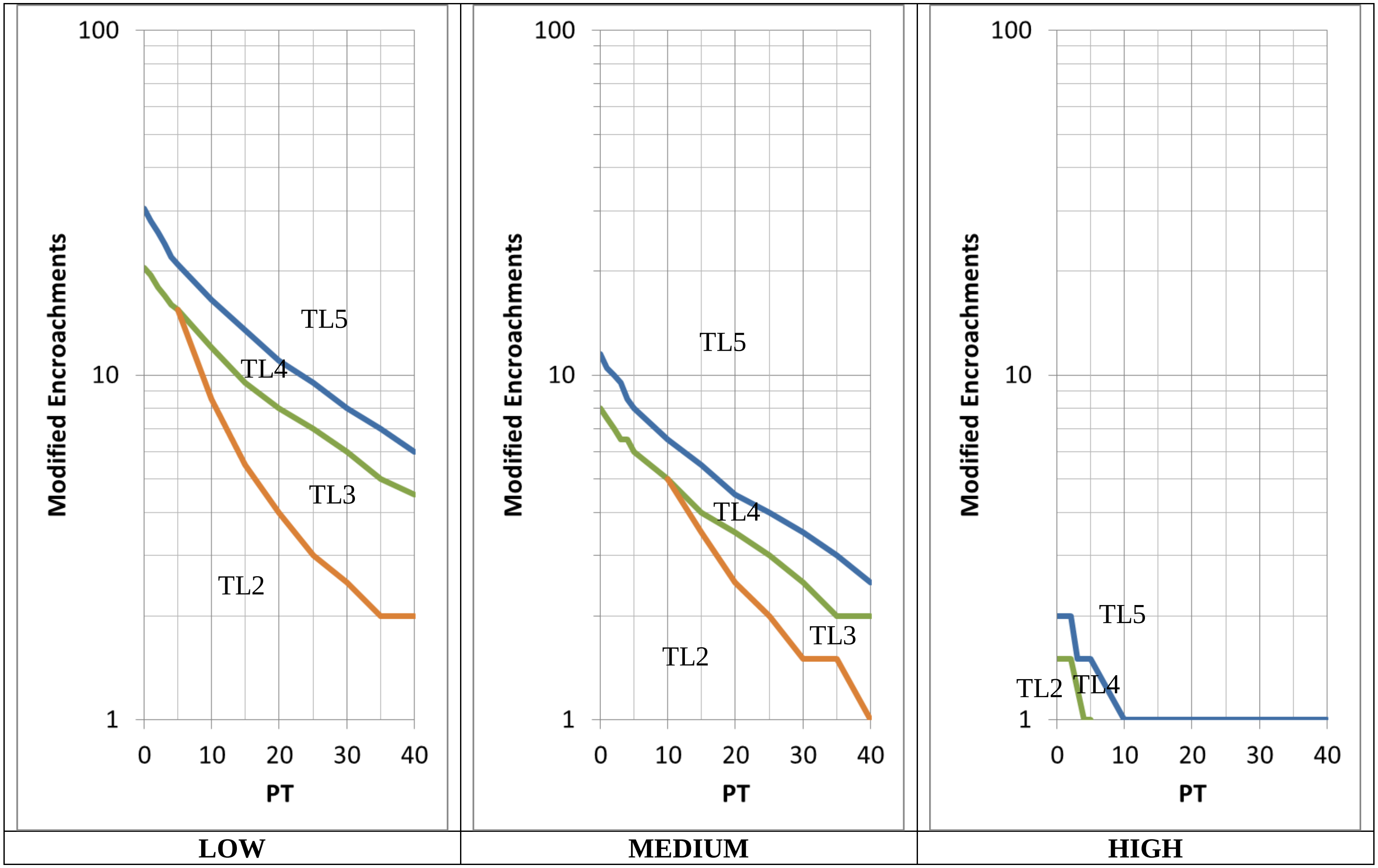


Figure 59. Test Level Selection Nomograph (Benefit-Cost Ratio=2).

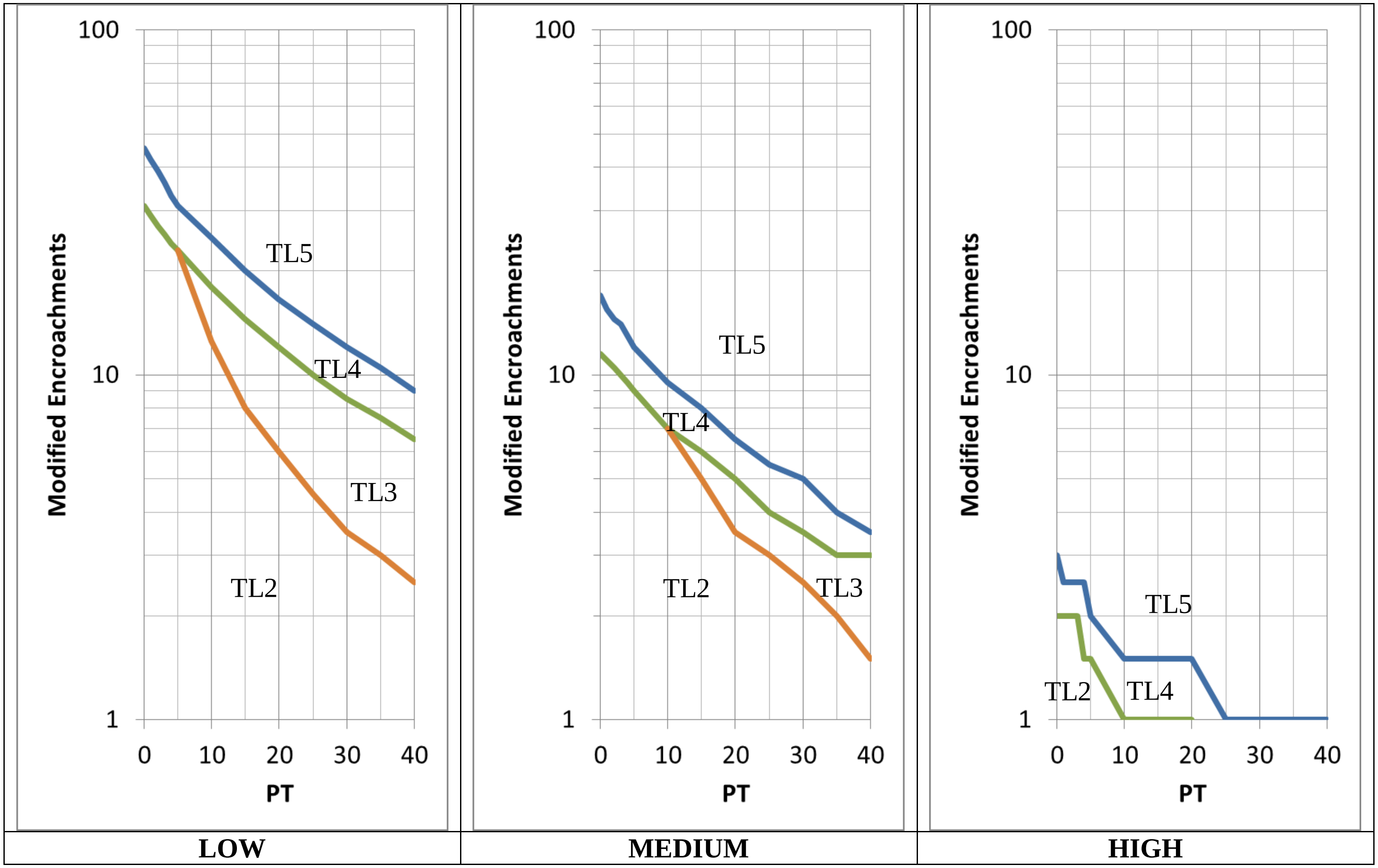
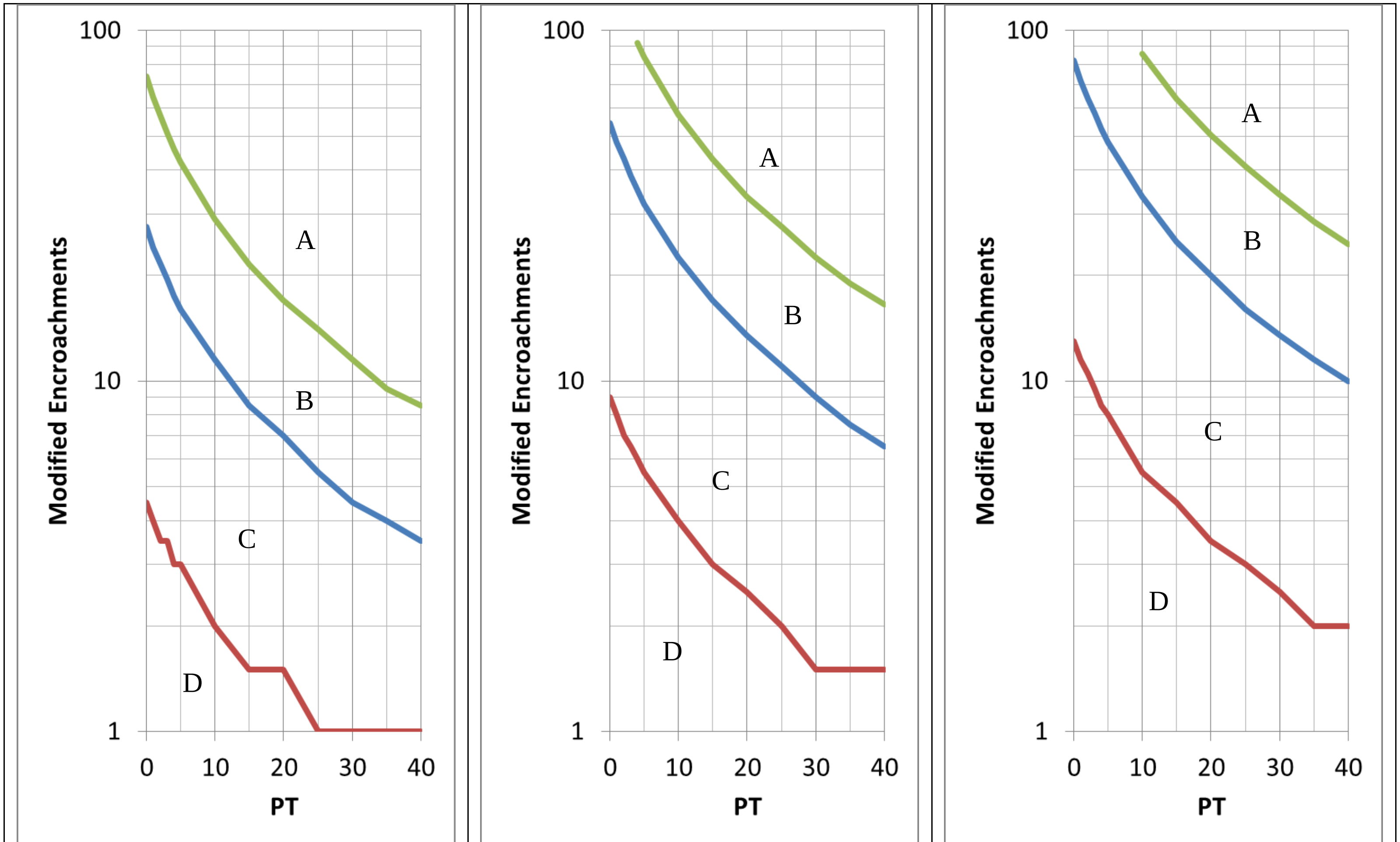


Figure 60. Test Level Selection Nomograph (Benefit-Cost Ratio=3).

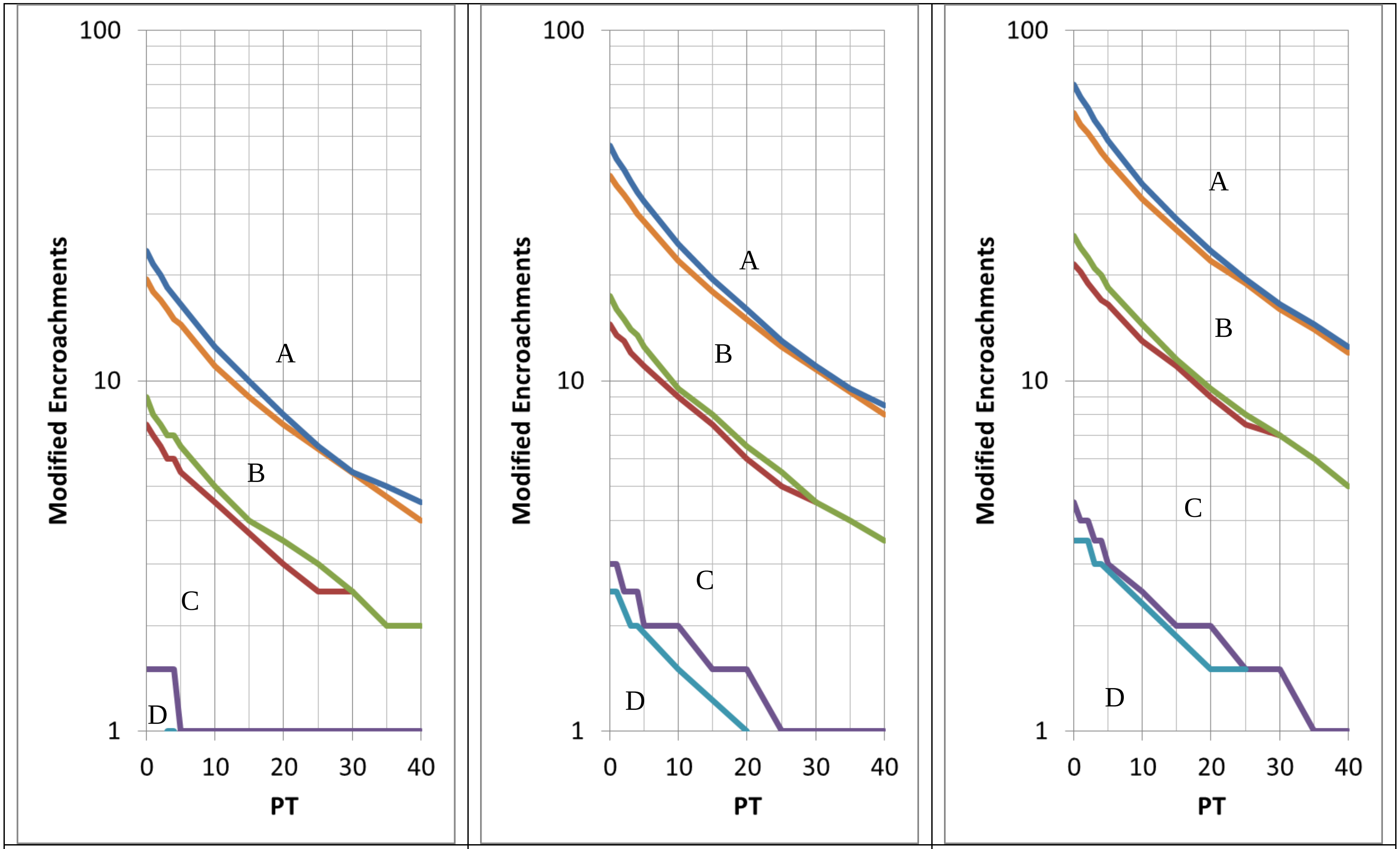


BCR=1 | BCR=2 | BCR=3

Figure 61. Rehabilitation Nomograph: Upgrade from R350 TL4 to MASH TL5.*

A: In this region, upgrade for all hazard levels.
B: In this region, upgrade for high or median hazard levels
C: In this region, upgrade for high hazard levels.
D: Do not upgrade in this region.

*NOTE: It is not cost-beneficial to upgrade from a R350 TL4 to a MASH TL4.

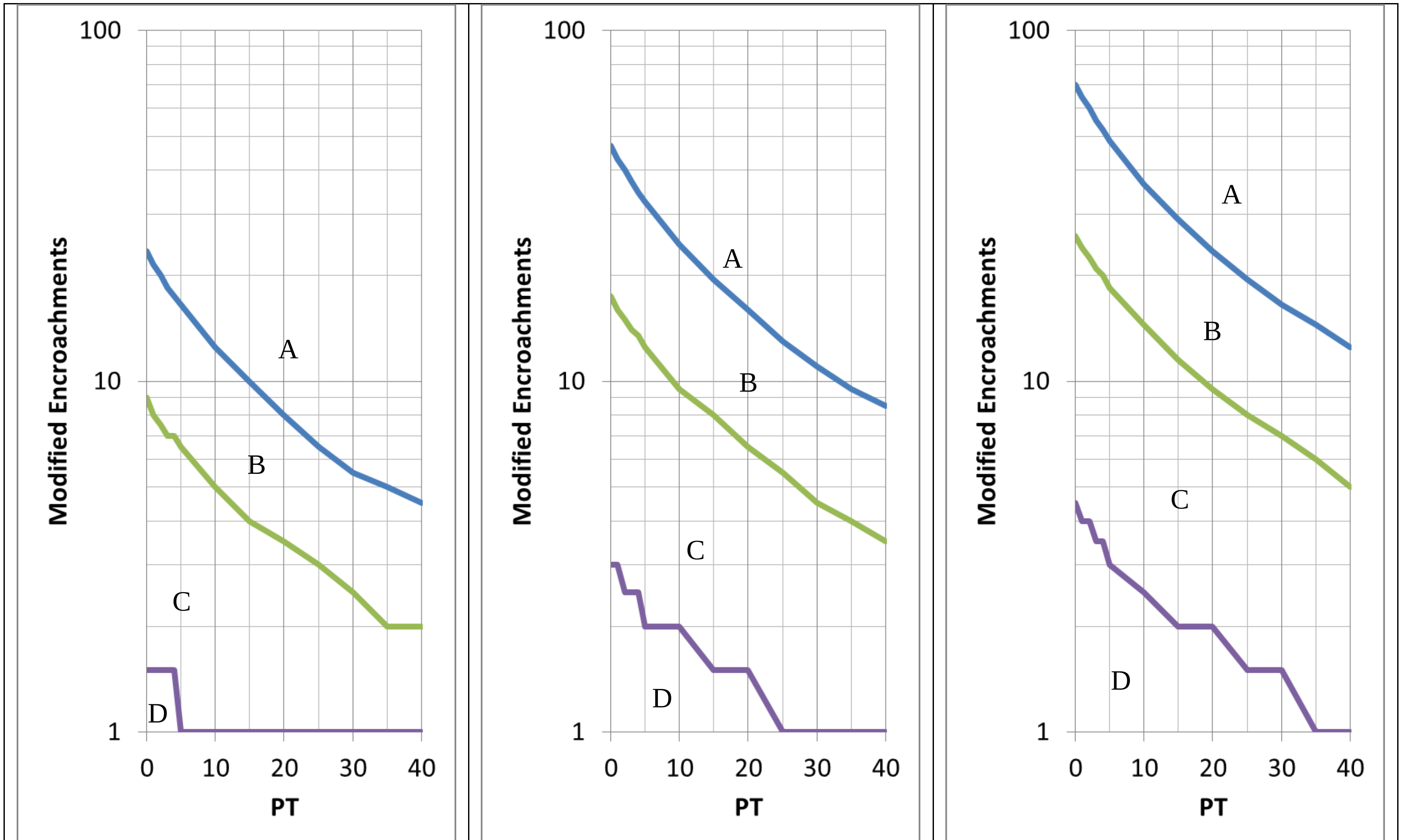


BCR=1	BCR=2	BCR=3

Figure 62. Rehabilitation Nomograph: Upgrade from R350 TL3 to MASH TL4 or MASH TL5.*

A: In this region, upgrade for all hazard levels.
B: In this region, upgrade for high or median hazard levels
C: In this region, upgrade for high hazard levels.
D: Do not upgrade in this region.

*NOTE: When a double line is present, the bottom line represents when it is cost-beneficial to upgrade to a TL4 barrier, where the top line represents when it is cost-beneficial to upgrade to the TL5 barrier.



BCR=1	BCR=2	BCR=3

Figure 63. Rehabilitation Nomograph: Upgrade from R350 TL3 to MASH TL5.

A: In this region, upgrade for all hazard levels.
B: In this region, upgrade for high or median hazard levels
C: In this region, upgrade for high hazard levels.
D: Do not upgrade in this region.

APPENDIX B: BRIDGE RAIL SELECTION GUIDELINES

The following section presents the recommended selection guidelines and the process for the application of the guidelines for the selection of MASH TL2 though TL5 bridge railings. A risk approach applicable to new construction, rehabilitation and retrofitting is recommended. An alternative cost-benefit approach is provided and discussed following the presentation of the process. A discussion of the process and the policy decisions necessary for implementation are also contained later in this section.

Appendix B includes only the process, without any discussion. Appendix B is presented in a format that could be inserted directly into the AASHTO LRFD Bridge Design Specification or the Roadside Design Guide.

Bridge Rail Risk Assessment Process

The selection of the appropriate MASH test level bridge railing for new or rehabilitation construction is dependent on site-specific conditions and results may differ for each side of the bridge. This process, therefore, should be followed for each bridge edge.

These selection procedures only apply to the bridge railing itself. Providing appropriate guardrail-bridge rail transitions, adequate guardrail approaches, and appropriate terminals and crash cushions are also important considerations in the complete safety performance of the bridge. Users should refer to the AASHTO Roadside Design Guide for guidance on appropriate transitions, approach guardrails and terminals.

The following selection guidelines include six parts: (1) determine the anticipated construction year traffic volume (AADT); (2) estimate the total encroachments expected over the 30-year life of a 1,000-ft section of the bridge; (3) adjust the expected number of encroachments for site-specific conditions; (4) select the test level from the appropriate chart; (5) additional considerations; and (6) if guidelines do not apply. These steps are described in full below.

7. Traffic Conditions – Determine the anticipated construction year traffic volume (AADT) and percent trucks (PT). These selection guidelines assume an annual traffic growth rate of 2% per year and a design life of 30 years. If the anticipated growth rate or design life are significantly different, use the following equation to compute the equivalent construction year traffic volume for use in these selection guidelines:

$$AADT_{EQ} = 0.7430 \cdot AADT_0 \cdot (1+G)^{L/2}$$

where:

$AADT_0$	= The anticipated construction year bi-directional traffic volume (use the one-way traffic volume for one-way roads and ramps),
$AADT_{EQ}$	= The equivalent construction year bi-directional traffic volume,
G	= The anticipated annual traffic growth rate where $0 \leq G \leq 1$.
L	= The design-life of the bridge railing in years.

8. Encroachments – Estimate the total number of encroachments (N_{ENCR}) that will be experienced on a 1,000-ft section of the bridge railing during the life of the bridge railing by entering Table 68 or Figure 50 with the bi-directional construction year AADT from Step 1 and the highway type.
    a. Do not proportion the value of N_{ENCR} based on the length of the bridge. The entire method is based on a per 1,000-ft basis.
    b. If the AADT of interest falls to the right of the end of the curve for the desired highway type, the level of service for the highway is likely D or worse and these procedures cannot be used; refer to step 6.

9. Site Conditions – Determine the site-specific adjustment factors for the bridge under consideration using the adjustment factors shown in Table 67. Multiply all the adjustments from Table 67 together to obtain f_{TOT}. Find the modified total number of encroachments ($N_{MOD\ ENCR}$) either by:
    a. Drawing a horizontal line in Figure 50 until the curve corresponding to f_{TOT} is obtained (interpolation between lines is acceptable) then reading down to the horizontal axis for the value of the modified total number of encroachments ($N_{MOD\ ENCR}$) on 1,000-ft of bridge railing over the 30-year life of the bridge railing or
    b. Multiplying the estimated encroachments (N_{ENCR}) from Table 68 by the total adjustments (f_{TOT}) from Step 2 to obtain the modified total number of encroachments ($N_{MOD\ ENCR}$) on 1,000-ft of bridge railing over the 30-year life of the bridge railing.

10. Test Level Selection – Characterize the hazard environment under the bridge as *high*, *medium* or *low* according to the following definitions:

HIGH: A high hazard environment below the bridge includes possible interruption to regional transportation facilities (i.e., high-volume highways, transit and commuter rail, etc.) and/or interaction with a densely populated area below the bridge. Penetrating the railing may limit or impose severe limitations on the regional transportation network (i.e., interstates, rail, etc.). Penetrating the railing also has the possibility of causing multiple fatalities and injuries in addition to the injuries associated with the vehicle occupants. A high-hazard environment is also present if penetration or rolling over the bridge railing could lead to the vehicle damaging a critical structural component of the bridge (e.g., a through-truss bridge).

MEDIUM: A medium hazard environment below the bridge includes possible interruption to local transportation facilities, large water bodies used for the shipment of goods or transportation of people, and/or damage to an urban area which is not densely populated. Penetrating the railing would limit local transportation routes, however, detours would be possible and reasonable. Penetrating the railing has the possibility of causing at least one non-motor vehicle injury or fatality.

LOW: A low hazard environment below the bridge includes water bodies not used for transportation, low-volume transportation facilities, or areas without buildings or houses in the vicinity of the bridge. Penetrating a low hazard railing would have little impact on regional or local transportation facilities. A low hazard railing has no buildings or facilities in the area which present possible non-motor vehicle related victims of a rail penetration.

Choose the hazard environment most applicable to the bridge under consideration. Enter the appropriate chart in Figure 52 for the hazard environment selected above, the modified lifetime encroachments per 1,000-ft of bridge edge ($N_{MOD\ ENCR}$) from Step 3, and the percent trucks (PT) from Step 1 to select the appropriate MASH test level for the bridge railing. If the point plots above the dashed risk boundary these charts cannot be used and the engineer should refer to step 6.

11. Additional Considerations – The bridge railing selected using this process provides a solution where the risk of observing a severe or fatal injury crash over the design-life of the bridge railing should be less than 0.01 when the specific site conditions evaluated (i.e., traffic volume and mix, geometry, posted speed limit, and access density) are considered. Engineering judgment should be used when unusual or difficult to characterize site conditions are encountered when selecting a bridge railing. Limited numbers of crash tested bridge railings are available at some test levels, therefore, it is possible that the recommended test level barrier for the evaluated site conditions may not be the best choice for some site conditions not explicitly addressed in these selection guidelines. For example, the particular layout of the barrier at the end of a ramp may influence intersection sight distances and require the use of engineering judgment in designing the interchange to determine an appropriate barrier as it approaches the intersection. Another example might be the presence of pedestrians or bicyclists which might benefit from a higher or different type of railing or the use of sidewalks. Some of the factors that should also be considered are:
    a. TL5 bridge railings may be appropriate for specially designated hazardous material or truck routes.
    b. Intersection sight distance obstructions created by higher test level bridge railings at the ends of ramps or bridges should be considered and the bridge railings may require transitioning to a lower height approaching the intersection.
    c. Stopping sight distance on bridges where the radius and design speed plot below the dashed line in Figure 51 may limit the use of higher test level bridge railings.
    d. The presence of pedestrians, bicyclists, snowmobiles, all-terrain vehicles and other recreational vehicles may affect the choice of bridge railing.

e. Crash history especially as it relates to heavy vehicle crashes or bridge rail penetrations may justify higher performance bridge railings.
f. Regional concerns about snow removal, hydrological impact of flood waters flowing over the bridge, and maintaining scenic views may also play a role in the selection of bridge railings beyond these selection guidelines.
g. The capacity of the bridge deck may limit the choices available for higher test level bridge railings on rehabilitation projects.

12. Guidelines Do Not Apply – There are some situations where these guidelines should not be used, namely:

    a. The traffic conditions violate the free traffic flow assumption used in developing the guidelines such that the estimate of the number of encroachments is not reliable. Generally, this results from a plot point in Figure 50 that is to the right of the end of the highway-type line. This indicates that the level of service may be D or worse and the basic assumptions of the method are invalid.
    b. The user may find that the selection plots above the boundary of Figure 52. In such a case the following options should be considered:
        i. Can the traffic operational conditions (i.e., AADT and percent trucks) be reduced?
        ii. Are the roadway characteristics (e.g., horizontal curvature, grade, etc.) resulting in large adjustments to the N_{ENCR}? Can the geometry be modified to reduce the adjustments?
        iii. Can the deck and superstructure support a TL6 bridge railing?

    These situations require a more detailed analysis of the site conditions that examines a broader range of alternatives beyond just the bridge railing test level selection. A solution will probably require the collaboration of traffic operations, geometric design and bridge railing design engineers to either modify the traffic or geometry conditions of the bridge such that these guidelines can be used or perform a crash history investigation to determine the actual performance of the existing bridge railing.

Table 67. Encroachment Adjustments.

Access Density			Lane Width			Horizontal Curve Radius	
Number of Access Points on Bridge or within 200 ft of either end	Undivided	Divided and Oneway	Avg. Lane Width in feet	Undivided	Divided and Oneway	Horizontal Curve Radius at Centerline in feet	All Highway Types
0	1.0	1.0	9≥	1.50	1.25	$950 \geq R_L$	4.00
1	1.5	2.0	10	1.30	1.15	$1910 > R_L > 950$	$(2228.2 - R_L)/318.3$
2≤	2.2	4.0	11	1.05	1.03	$1910 \leq R_L$	1.00
			12≤	1.00	1.00	$950 \geq R_R$	2.00
						$1910 > R_R > 950$	$(R_R - 2864.8)/954.9$
						$1910 \leq R_R$	1.00
f_{ACC}=			f_{LW}=			f_{HC}=	

Lanes in One Direction			Posted Speed			Grade	
No. of Through Lanes in One Direction	Undivided	Divided and Oneway	Post Speed Limit	Undivided	Divided and Oneway	Percent Grade	All Highway Types
1	1.00	1.00	<65	1.42	1.18	$-6 \geq G$	2.00
2	0.76	1.00	≥65	1.00	1.00	$-6 < G < -2$	$0.5 - G/4$
3≤	0.76	0.91	For roads with unposted speed limits use the adjustment for <65 mi/hr.			$-2 \leq G$	1.00
f_{LN}=			f_{PSL}=			f_G=	

$f_{TOT} = f_{ACC} \cdot f_{LN} \cdot f_{LW} \cdot f_G \cdot f_{HC} \cdot f_{PSL}$ =

Table 68. AADT – Lifetime Encroachments per 1,000-ft of Bridge Railing.

AADT	4 LN DIV	2 LN UNDIV	1 LN ONEWAY
500	0.8	1.2	1.7
1,000	1.6	1.9	3.4
2,000	3.1	3.2	6.0
3,000	4.4	3.6	8.1
4,000	5.5	3.6	9.6
5,000	6.5	3.4	10.6
6,000	7.4	3.1	11.4
7,000	8.1	2.7	11.8
8,000	8.8	2.3	12.0
9,000	9.3	2.0	12.0
10,000	9.7	1.9	11.9
11,000	10.1	1.8	11.7
12,000	10.4	1.8	11.6
13,000	10.6	1.8	
14,000	10.8	1.9	
15,000	10.9	2.0	
16,000	11.0	2.1	
17,000	11.0	2.2	
18,000	11.0	2.4	
19,000	10.9	2.5	
20,000	10.9	2.6	
21,000	10.8	2.7	
22,000	10.7	2.9	
23,000	10.6	3.0	LOS ≥ D
24,000	10.6	3.1	
25,000	10.5	3.3	
26,000	10.6	3.4	
27,000	10.7	3.5	
28,000	10.8	3.7	
29,000	10.9	3.8	
30,000	11.0	3.9	
31,000	11.2	4.1	
32,000	11.4	4.2	
33,000	11.6	4.3	
33,000	11.6	4.3	
34,000	11.8	4.4	
35,000	12.1	4.6	
36,000	12.4	4.7	
37,000	12.6	4.8	
38,000	12.9	5.0	
39,000	13.2	5.1	
40,000	13.5	5.2	
41,000	13.9	5.4	
42,000	14.2	5.5	
43,000	14.5	5.6	
44,000	14.9	5.8	
45,000	15.2	5.9	
46,000	15.6	6.0	
47,000	15.9		
48,000	16.2		
49,000	16.6		LOS ≥ D
50,000	16.9		
51,000	17.2		
52,000	17.6		
53,000	17.9		
54,000	18.3		
55,000	18.6		
60,000	20.3	LOS ≥ D	
65,000	22.0		
70,000	23.7		
75,000	25.4		
80,000	27.1		
85,000	28.7		
90,000	30.4		
95,000			
100,000	LOS ≥ D		
105,000			
110,000			

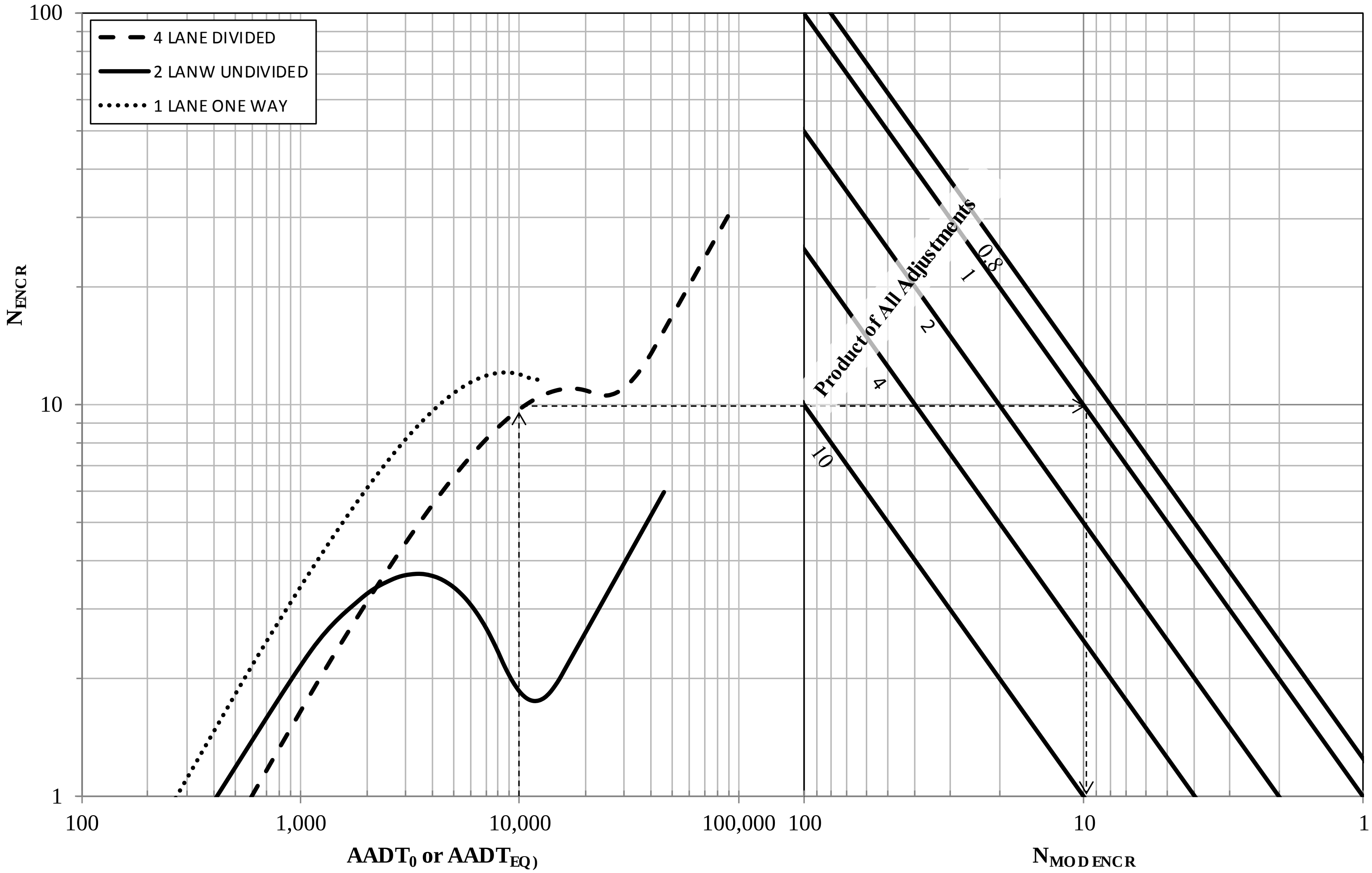


Figure 50. AADT – Lifetime Encroachments/1,000-ft of Bridge Railing Nomograph.

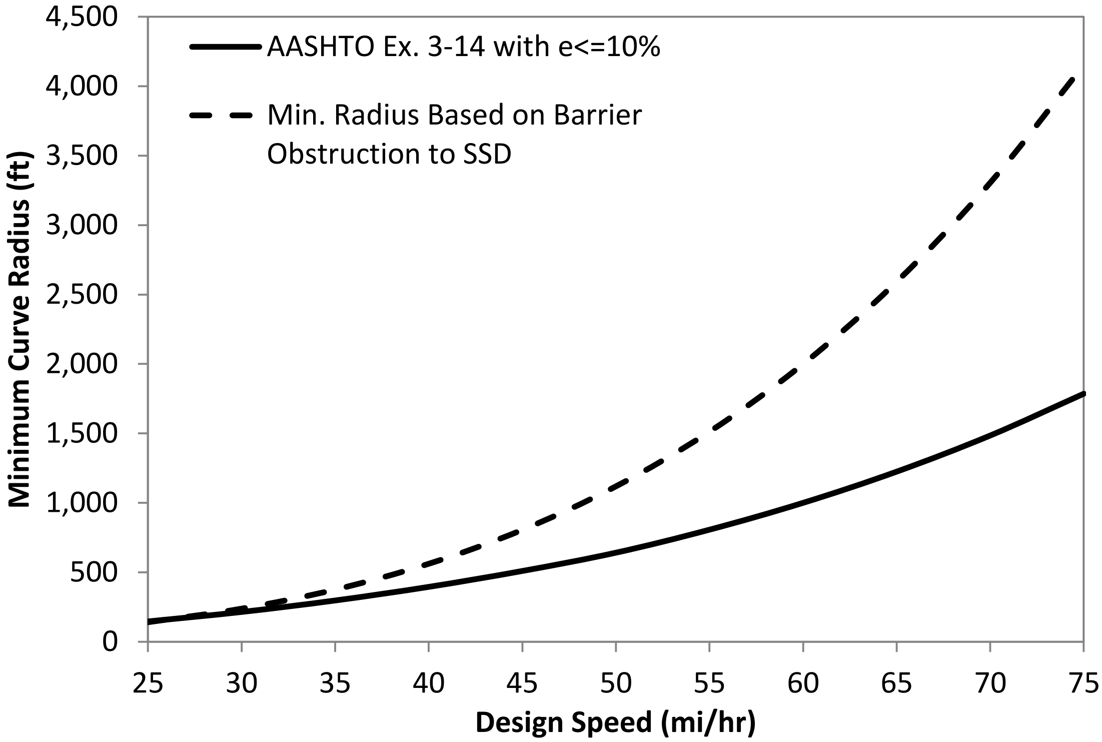


Figure 51. Minimum Horizontal Curve Radius Based on Barrier Obstruction to the Stopping Sight Distance Compared to AASHTO Exhibit 3-14.

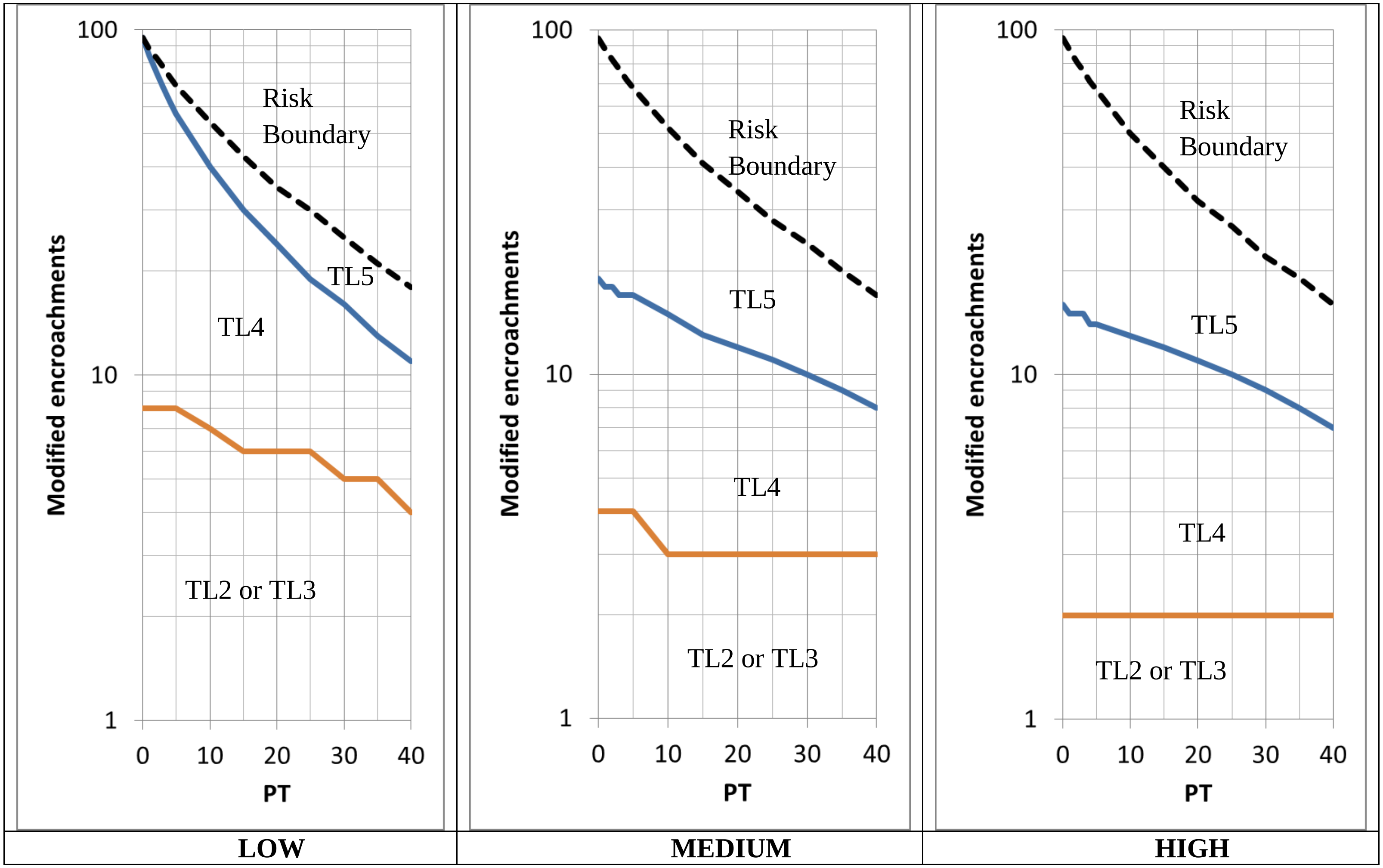


Figure 52. Test Level Selection Nomograph (Risk<0.01 in 30 years for 1000 ft of bridge railing).

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