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Development of a Bridge Railing Optimized for Rural, Low-Volume Roads

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Abstract

A new steel bridge rail was developed for use on rural, low-volume bridges. The railing consisted of 31-in. tall, 12-gauge W-beam guardrail mounted on S3x5.7 posts, which were supported by steel square-tube sockets. These side-mounted sockets were attached to the deck edge using a unique bolted design that connects directly to coupling nuts and threaded anchor rods embedded into the bridge deck. Thus, during a crash, the tensile impact loads are transferred directly to the anchor rods and the risk of damage to the deck edge is minimized. Full-scale crash testing was conducted according to test 2 to 11 of the AASHTO's *Manual for Assessing Safety Hardware* (MASH). The test vehicle struck the bridge rail at 44.2 mph and an angle of 25.5° and was successfully contained and redirected. Damage to the bridge rail consisted of bent posts and deformed guardrail. No damage to the deck or sockets was observed. The tests passed all evaluation criteria of MASH test 2 to 11. The new railing was deemed crashworthy to MASH Test Level 2 (TL-2) with a post spacing of 75 in. Additionally, when the post spacing is reduced to 37.5 in., the railing was determined to be MASH Test Level 3 (TL-3) crashworthy through a comparison to similar, crash-tested, W-beam guardrail systems. BARRIER VII simulations showed that the new railing could be directly connected to the Midwest Guardrail System without a transition. Guidance was provided pertaining to the length of guardrail required adjacent to the bridge rail.

Keywords

infrastructure, roadway design, low-volume roads, bridges and stream crossings, safety, roadside safety design, guardrail, MASH

Late in the 2010s, the Nebraska Department of Transportation (NDOT) launched a voluntary county bridge match-assistance program intended to aid Nebraska counties in replacing deteriorated bridges. The program targeted rural, two-lane bridges on low-volume roadways characterized as having only 50–1000 vehicles per day. As part of this program, NDOT wanted to ensure that the bridge railings on these new bridges were both cost-effective and crashworthy to the safety performance criteria of AASHTO's *Manual for Assessing Safety Hardware* (MASH) (*1*).

A survey of bridge railing designs commonly used throughout the Midwest revealed numerous designs utilizing corrugated steel rails supported by steel posts. Examples of these bridge railing designs are shown in Figure 1. However, several safety issues were identified with these common railing designs:

- No guardrail anchorage: Corrugated guardrail systems such as W-beam and thrie-beam barriers rely on a combination of post strength and membrane action to perform properly. Very little of a guardrail installation's strength comes from the bending of the guardrail itself. Rather, as the barrier deflects backward, the deformed guardrail is stretched, and a combination of the tension built in the rail and the angles formed by the deformed

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Figure 1. Common steel post-and-beam bridge railings found on rural, low-volume roads.

rail shape results in lateral loads that redirect vehicles. This membrane action is similar to person walking a tightrope. Proper guardrail anchors are required on both ends of an installation, to develop the tensile forces in the guardrail. Without guardrail anchors, the rail will simply be pulled from the posts, allowing the vehicle to traverse the barrier.

- Guardrail blunt ends: If corrugated guardrail installations are struck end-on, the guardrail end can slice straight through a vehicle. To prevent this from happening, guardrail installations require end treatments that not only anchor the guardrail, but also prevent it from spearing the vehicle. Applying short, flattened rail segments that curve away from the roadway, commonly referred to as "spoons" or "fish tails" (shown in Figure 1), is not enough to prevent vehicle spearing.
- Inadequate length of need: As explained in the AASHTO's *Roadside Design Guide* (*2*), guardrail length of need is required to extend upstream of any hazard, to properly shield the hazard. However, the guardrail bridge railings found on many rural, low-volume roads did not extend off the bridge. A vehicle exiting the roadway just before the bridge would easily miss the railing and end up at the bottom of the feature the bridge was crossing.
- Posts as snag risks: Many of the bridge rail posts described in the survey were W6x15, W6x20, or larger steel sections. Vehicle contact with posts of this size and strength often results in the vehicle snagging on the posts and leads to excessive decelerations and/or vehicle instabilities (i.e., abrupt stops, vehicle rollover, or both). The bridge railings described within the survey rarely contained blockouts to mitigate vehicle snag potential. Thus, the risk of vehicle snag on the bridge rail posts was significant.

Because of the safety issues observed with the typical bridge railings found on rural, low-volume roads, a new, MASH-crashworthy, cost-efficient bridge railing was desired.

Background

NCHRP project 22-12(03) provided guidelines for the selection of bridge rails based on roadway characteristics

Figure 2. Midwest Guardrail System bridge rail test installation.

Figure 3. Texas Department of Transportation's T631 test installation.

such as traffic volume, percentage of heavy trucks, speed, lane width, curvature, and perceived risk of a railing failure (*3*). In general terms, it was found that a Test Level 2 (TL-2) system would be warranted for nearly all roadways with a traffic volume less than 1000 vehicles per day because of the low risk of vehicle encroachment. Test Level 1 (TL-1) barriers were not considered in the NCHRP analysis. However, the cost difference between a TL-1 and a TL-2 system is often minimal. Thus, a TL-2 bridge rail would cover all bridges within the NDOT program with negligible cost increases for the very low-volume roads.

Two W-beam bridge rails have been designed for use on low-volume roads that satisfy MASH TL-2 and Test Level 3 (TL-3) safety standards. Both systems utilized a 31-in. tall W-beam rail supported by S3x5.7 weak posts. These weak-post systems absorbed impact energy through plastic bending of the posts. Thus, the weak posts limited the loads transferred to the bridge deck and reduced the risk of deck damage as compared with a strong-post system.

The first railing, the Midwest Guardrail System (MGS) bridge rail, was a side-mounted system that was supported by steel sockets placed adjacent to the side of the deck (*4*, *5*), as shown in Figure 2. The system utilized a 37.5-in. post spacing, and the sockets were attached to the bridge deck with a 1-in. diameter bolt that went through the thickness of the deck. A steel angle was mounted below the deck to provide additional length for the force couple, which resisted post bending. The MGS bridge rail was full-scale crash tested and satisfied MASH TL-3 criteria.

The second railing, the Texas Department of Transportation's (TxDOT) T631 bridge rail, was a top-mounted system that utilized an S3x5.7 post and $^5/_8$-in. thick base plate. The post assemblies were bolted to the top of the bridge deck with four $^5/_8$-in. diameter bolts (*6*, *7*), as shown in Figure 3. With a 75-in. post spacing, the T631 was successfully crash tested to MASH TL-2 criteria, but failed MASH TL-3 because of rail rupture. A modified version of the system with a 37.5-in. post spacing was later crash tested and satisfied MASH TL-3 criteria.

These existing railings required attachment hardware on the top surface of the bridge deck, which occupied deck width and placed obstructions on the deck surface. However, it was believed that a similar bridge railing could be developed as a purely side-mounted railing, thereby freeing up deck space and further optimizing the design.

Objective

The research objectives for this project included the development and full-scale crash testing of a TL-2 bridge railing for use on rural, low-volume roadways. A railing incorporating side-mounted posts was desired to limit encroachment of the system over the bridge deck and maximize the traversable width of the bridge. The bridge railing was to be compatible with both cast-in-place (CIP) decks as well as precast beam slabs and minimize deck damage during impact events. An analysis of the guardrail requirements adjacent to the bridge was also desired to limit total installation costs.

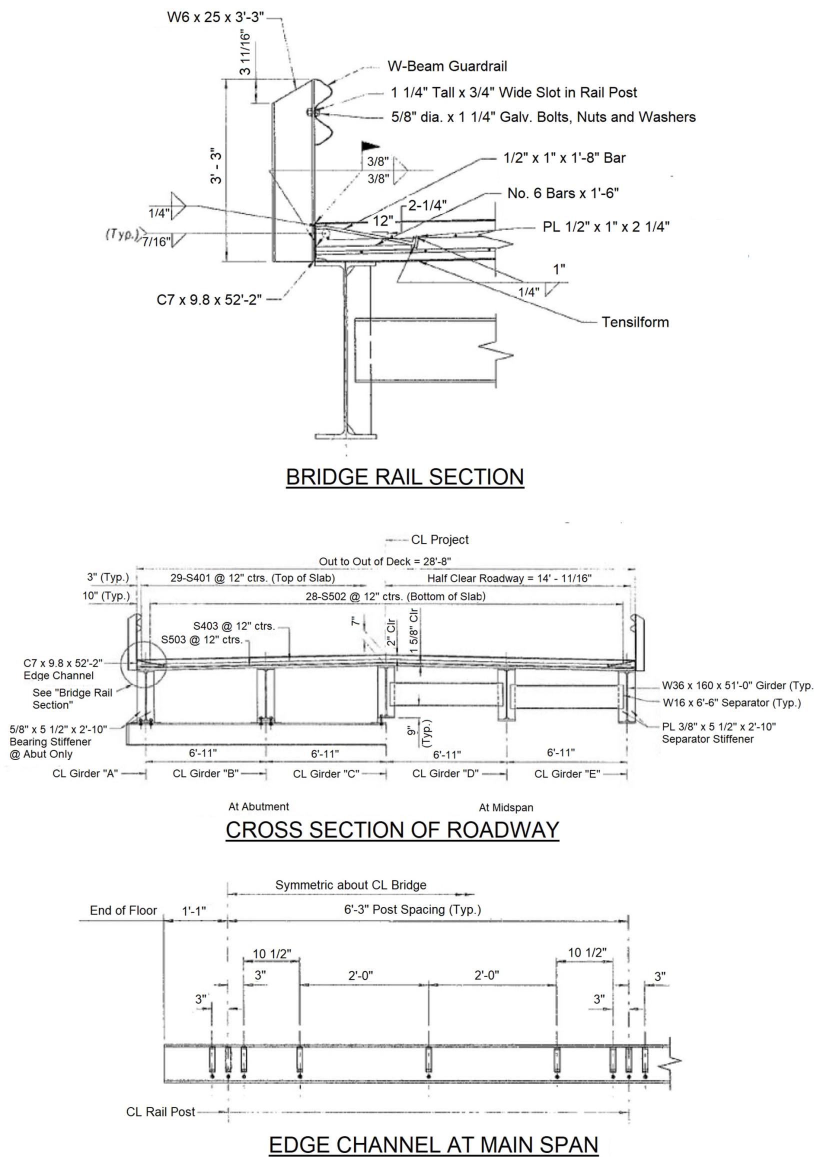

Figure 4. Example details for 7-in. thick cast-in-place deck.

Design Details

Deck Configurations

Historically, bridges on rural, low-volume roads in the state of Nebraska were built with CIP decks. However, the use of precast, prestressed beam slabs has become popular in recent years as they allow for rapid bridge construction. Thus, NDOT desired the new bridge railing to be compatible with both deck types.

CIP decks in Nebraska are typically 7- or 8-in. thick and are reinforced with upper and lower steel rebar mats. These decks are supported by wide-flange steel girders, and the exterior girders are commonly placed directly below the edges of the deck. As such, there are no overhang or cantilevered portions of deck along the sides of the bridge. Steel channels are commonly placed along the edges of the deck. These channels are tack welded to the tops of the exterior girders and serve as formwork while pouring the deck. Additionally, the channels provide a steel surface along the edge of the deck where bridge rail posts can be welded onto the bridge. Rebar are welded to the inside face of the channel and tied to both the top and bottom steel mats to anchor the channels to the side of the deck. Example details from a typical CIP bridge deck are shown in Figure 4.

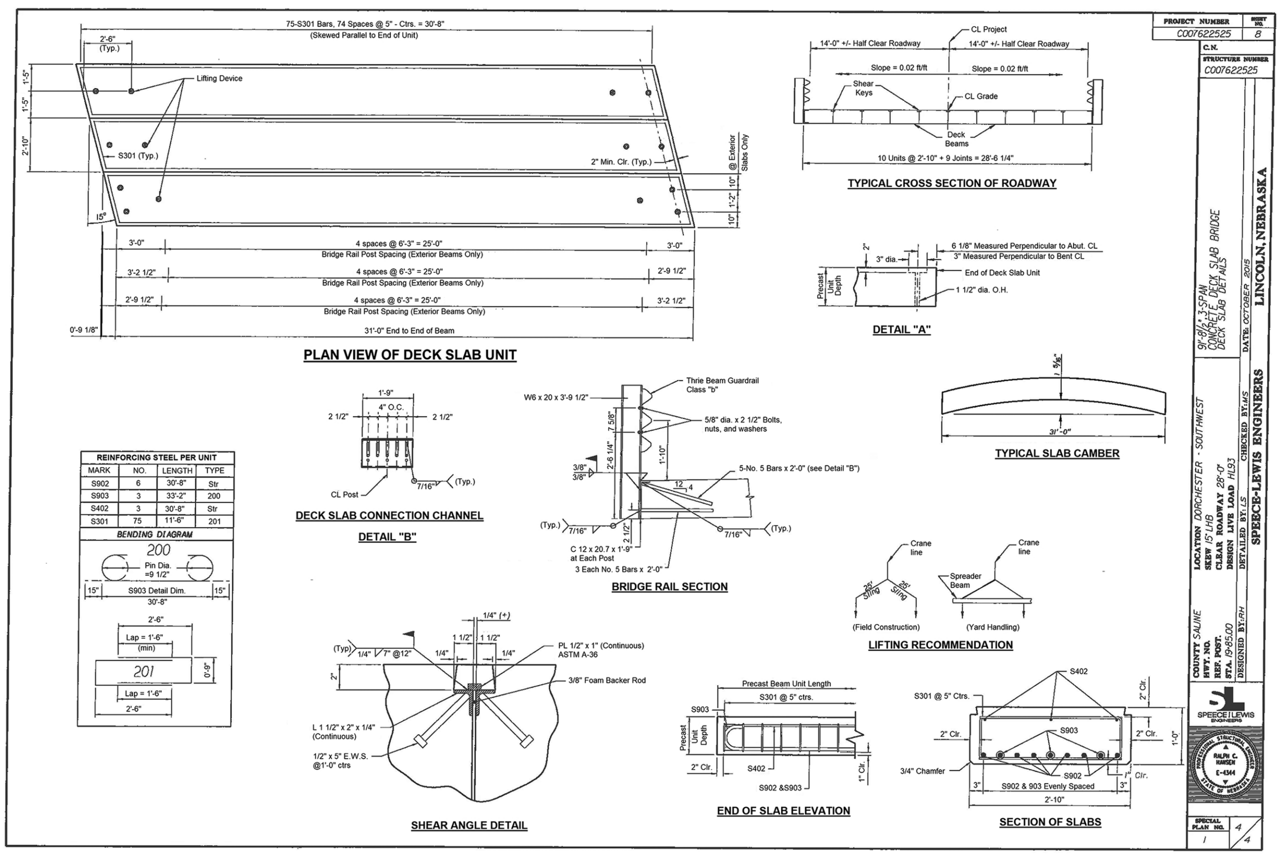

Figure 5. Example details for 12-in. thick precast beam-slab deck.

Through discussions with NDOT, a 7-in. thick deck reinforced with #4 lateral rebar at 6-in. spacings and #4 longitudinal rebar at 12-in. spacings in both the upper and lower steel mats was selected as a critical CIP deck for use in the development and evaluation of the new bridge railing. This configuration represented the thinnest CIP deck and utilized typical reinforcing steel. A C7x9.8 channel was selected for use along the deck edge as it was the weakest of the standard 7-in. channels.

Precast, prestressed beam slabs can be fabricated in a variety of sizes and configurations, but they have a minimum thickness of 12 in. and are typically around 3–4 ft wide. Similar to the CIP decks, steel channels would be embedded into the sides of the exterior precast beam slabs to provide a steel surface for the attachment of bridge rail posts. However, since the channels are not needed as formwork, the side channels in beam slabs may be continuous along the edge or used intermittently only at post locations. Example details from a typical beam-slab bridge are shown in Figure 5, and pictures of short channel segments used in a recent bridge deck are shown in Figure 6. Through discussions with NDOT, a 12-in. thick beam slab, reinforced with #3 stirrups at 5-in. spacings, three #4 longitudinal rebar at the top, a combination of prestressing strands and rebar at the bottom, and with a C12x20.7 side channel, was selected as the critically small/weak beam-slab configuration for use in development and evaluation of the new bridge rail.

Post-to-Deck Attachment

It was desired for the new bridge rail to be a side-mounted, weak-post system similar to the railings shown previously. A socketed post-to-deck attachment similar to the MGS bridge rail was desired for its ease of installation and replacement of damaged posts. As such, the same steel post and socket sections used in the MGS bridge rail were used for the new railing. The posts were S3x5.7 sections and the sockets were HSS4x4x$^{3}/_{8}$ sections. Additionally, the posts contained the same $^{1}/_{4}$-in. thick post standoffs, or shims, welded to the sides of the post. These standoffs created a tighter fit for the post within the socket and prevented posts from leaning to the side, as shown in Figure 7. The only difference between the MGS bridge rail post and the posts for the new bridge

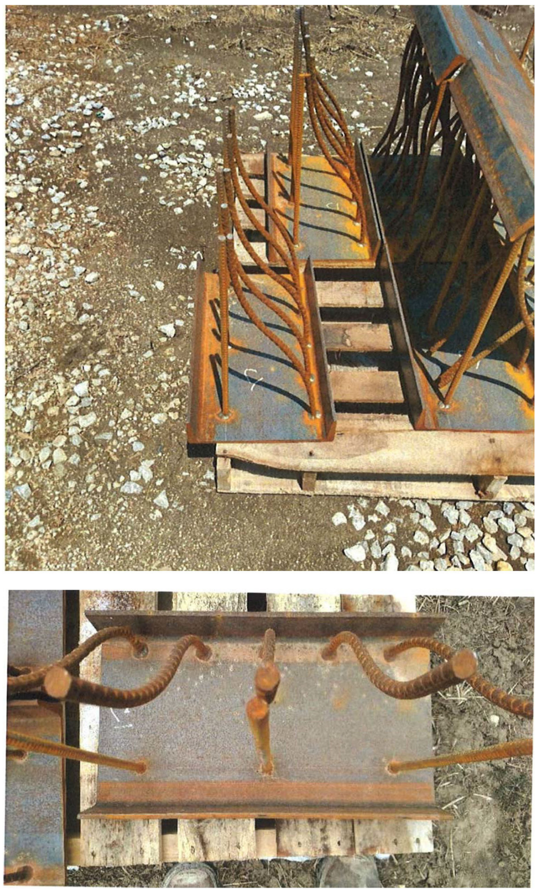

Figure 6. Short channel segments used within a 12-in. thick beam-slab deck.

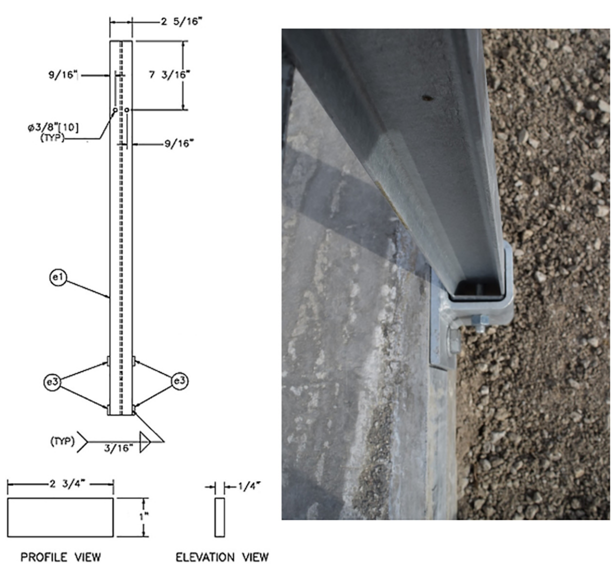

Figure 7. S3x5.7 weak post with $^{1}/_{4}$-in. thick standoffs.

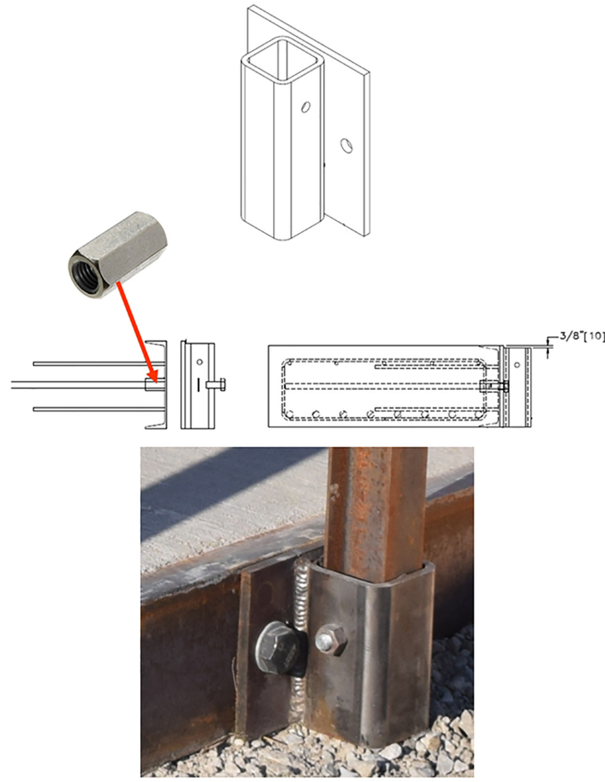

Figure 8. Coupling nut and threaded rod attachment of socket assembly to deck.

rail was the post length, which had to be adjusted to fit on the critical deck configurations.

A bolted socket-to-deck attachment was desired to eliminate field welding and to allow for quick and easy assembly of the railing on-site. However, the deck edge had to remain smooth and without hardware extending outward that would interfere with formwork. Thus, the socket assembly had to be bolted on from the outside with internally threaded components cast within the deck. To satisfy these constraints, an innovative post-to-deck attachment method was developed using coupling nuts and threaded anchor rods. Coupling nuts are commonly used to connect the ends of threaded hardware and directly transfer loads from one component to the other. For the new bolted attachment, holes were drilled in the web of the channel and coupling nuts were placed on the inside surface of the channel. Threaded rods were partially inserted into the coupling nuts and extended into the deck. These components would be embedded into the bridge deck during casting. Bolts would then be extended through the socket assembly's mounting plate and the side channel of the deck and be threaded into the coupling nut, as shown in Figure 8.

Figure 9. Dynamic testing of the post-to-deck attachment on 7-in. thick cast-in-place deck, post-test photographs.

Figure 10. Dynamic testing of the post-to-deck attachment on 12-in. thick beam-slab, post-test photographs.

During an impact, this new post-to-deck attachment design directly transfers the tensile loads from the attachment bolts through the coupling nuts and into the threaded rod anchors. The impact loads are never transferred to the channels (except for compression as part of the force couple resisting the moment created from post bending), so there was minimal risk of damage to the side channels or the deck. Finally, the coupling nuts, threaded rods, and bolts would all be standard hardware, so only the socket assembly consisting of the HSS socket and a mounting plate would need to be fabricated as part of the socket-to-deck attachment.

Dynamic component tests were conducted to evaluate the strength of this new post-to-deck attachment, with posts mounted to both critical deck configurations. Segments of both the 7-in. thick CIP deck and the 12-in. thick beam-slab deck were constructed with all of the previously noted internal reinforcement and the embedded coupling nuts and anchor rods. Short lengths of steel channel were placed at post locations along both deck edges instead of a continuous channel to represent a critical configuration. Anchor bolt, coupling nut, and anchor rod diameters were varied among the test articles.

Dynamic component testing consisted of an 1800-lb bogie vehicle striking the posts at a height of 25 in., a speed of 20 mph, and an angle of 90° relative to the roadway, or through the strong axis of the post. All the tests resulted in plastic bending of the post near the top of the socket. The new post-to-deck attachment design performed as intended as all of the tensile impact load was transferred through the coupling nuts and into the threaded anchor rods. No damage was observed to the concrete decks, the steel channels, the anchor bolts, or the steel socket assemblies. Post-test photographs from testing on the 7-in. thick CIP deck and the 12-in. thick beam-slab deck are shown in Figures 9 and 10, respectively. Based on these results, the new bridge rail design was recommended for full-scale crash testing.

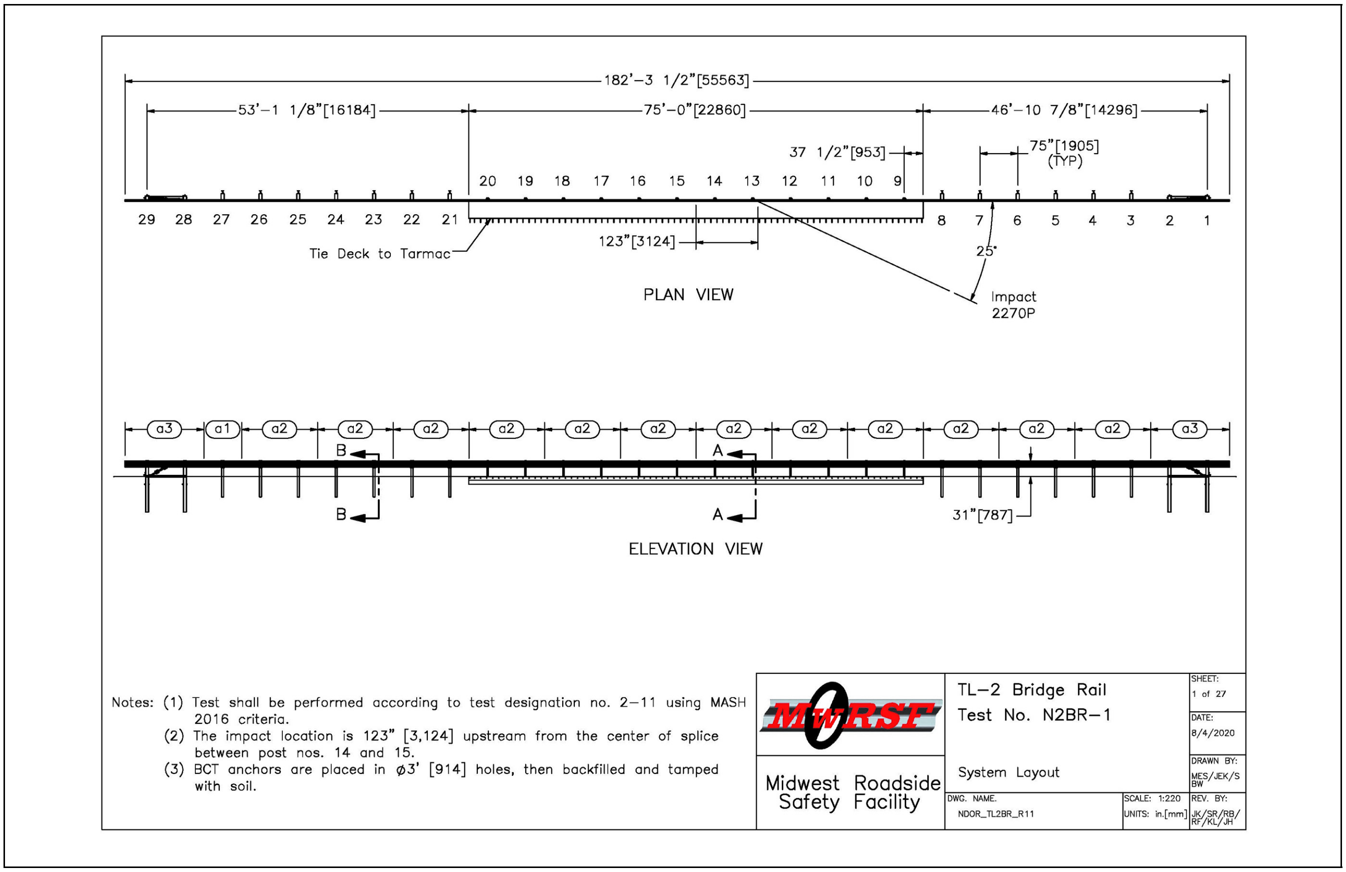

Figure 11. Test installation details.

Test Installation

A 75-ft long segment of the new bridge railing was placed in the middle of an 182-ft long MGS test installation, as shown in Figure 11. The bridge railing consisted of 31-in. tall, 12-gauge, W-beam guardrail supported by S3x5.7 posts spaced at 75 in. on-center. A $^{5}/_{16}$-in. diameter hex bolt and a $1^{3}/_{4}$-in. square washer were used to attach the guardrail to the posts, as shown in Figure 12. The side-mounted posts were inserted into socket assemblies consisting of HSS4x4x$^{3}/_{8}$ tube sockets and a $10 \times 7 \times$ ½-in. mounting plate. Standoff plates were welded to the bottom of the posts to create a tighter fit between the post and the socket and force the posts to stand vertical after installation. A 1-in. wide steel strap was welded to the bottom of each socket to prevent the post from falling through the socket during installation, and a $^{5}/_{8}$-in. diameter keeper bolt was used to prevent the post from pulling out of the socket during impact events.

The socket assemblies were attached to the deck using two $^{7}/_{8}$-in. diameter bolts that threaded into coupling nuts embedded into the side of the bridge deck, as shown in Figure 13. All-thread steel rods were threaded into the opposite side of the coupling nuts and extended into the simulated deck where they were secured to a $^{1}/_{4}$-in. thick plate washer. The mounting plates of the socket assembly contained vertical slots to allow for height adjustments of up to $^{1}/_{4}$-in. during installation, as shown in Figure 14.

The new bridge railing was to be compatible with both CIP and precast beam-slab decks. The 7-in. thick CIP deck was identified as the most critical deck configuration as it was the thinnest and weakest of the bridge decks, which made it more susceptible to damage and anchor pullout than the thicker and stronger decks. Thus, a simulated 7-in. thick CIP deck was selected for use in full-scale crash testing. The simulated 7-in. thick CIP bridge deck was 75 ft long, 36 in. wide, and was reinforced with #4 rebar in both the lateral and longitudinal directions for both the upper and lower steel mats. A C7x9.8 steel channel was cast into the outer edge of the deck. The channel assembly contained #4 rebar welded to the inside of its web that extended into the deck and tied into the upper and lower steel mats. The edge of the deck was supported by an unreinforced 8×12-in. grade beam meant to replicate an exterior bridge girder. The concrete's compressive strength was 5795 psi on test day.

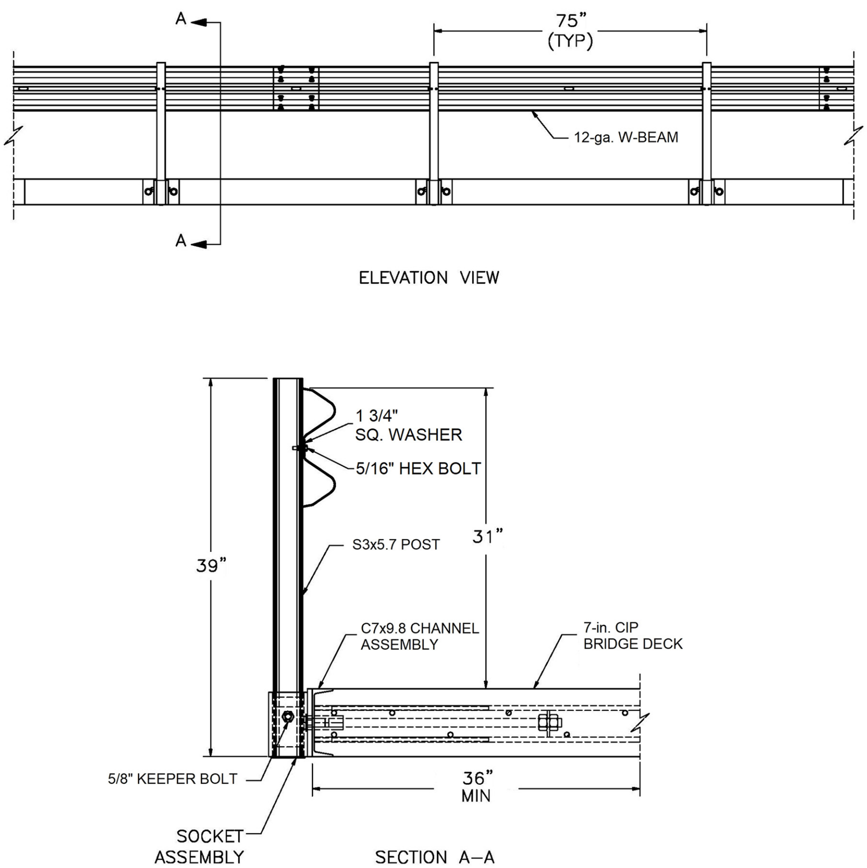

Figure 12. Bridge railing details.

Standard MGS, consisting of 31-in. tall W-beam guardrail and W6x8.5 posts spaced at 75 in. on-center, was installed on both sides of the bridge railing. The systems were connected with adjacent S3x5.7 bridge posts and W6x8.5 MGS posts spaced 75 in. apart. Thus, a constant post spacing was used throughout the entire test installation. The guardrail was anchored on both ends of the installation using a crashworthy trailing end terminal originally designed to simulate the strength of other crashworthy end terminals (*8*, *9*). Photographs of the test installation are shown in Figure 15.

Test Requirements and Evaluation Criteria

New roadside safety devices must satisfy the safety performance criteria of AASHTO's MASH (*1*) to be declared crashworthy. According to TL-2 of MASH, longitudinal barrier systems, such as bridge rails, are to be subjected to two full-scale vehicle crash tests, as summarized in Table 1.

Although MASH requires two full-scale crash tests, testing with the 1100C small car was not deemed critical for the evaluation of the new bridge rail. Previous MASH crash testing has been conducted with both the 2270P and the 1100C vehicles on the MGS bridge rail and the TxDOT T631 bridge rail (*4–7*). Similar to the new TL-2 bridge rail developed here, both of these previous bridge rails consist of 31-in. tall, 12-gauge, W-beam guardrail supported by S3x5.7 posts. Further, all three bridge rails were designed to absorb impact energy through bending of the S3x5.7 weak posts while the attachment of the post to the deck remains rigid and intact. The TxDOT T631 bridge rail was successfully tested to MASH tests 2-10 and 2-11 with a 75-in. post spacing, which is the same as the new TL-2 bridge rail. Additionally, the MGS bridge rail was successfully tested to MASH tests 3-10 and 3-11 with a 37.5-in. post spacing utilizing the same post assembly and HSS4x4x$^3/_8$ steel sockets incorporated into the new TL-2 bridge rail. Thus, if the socket assembly remained undamaged and intact throughout an impact event, the new TL-2 bridge rail would be expected to perform very similarly to the TL-2 version of the TxDOT T631. The increased mass of the 2270P test vehicle results in a higher impact severity, higher impact loads, and higher system

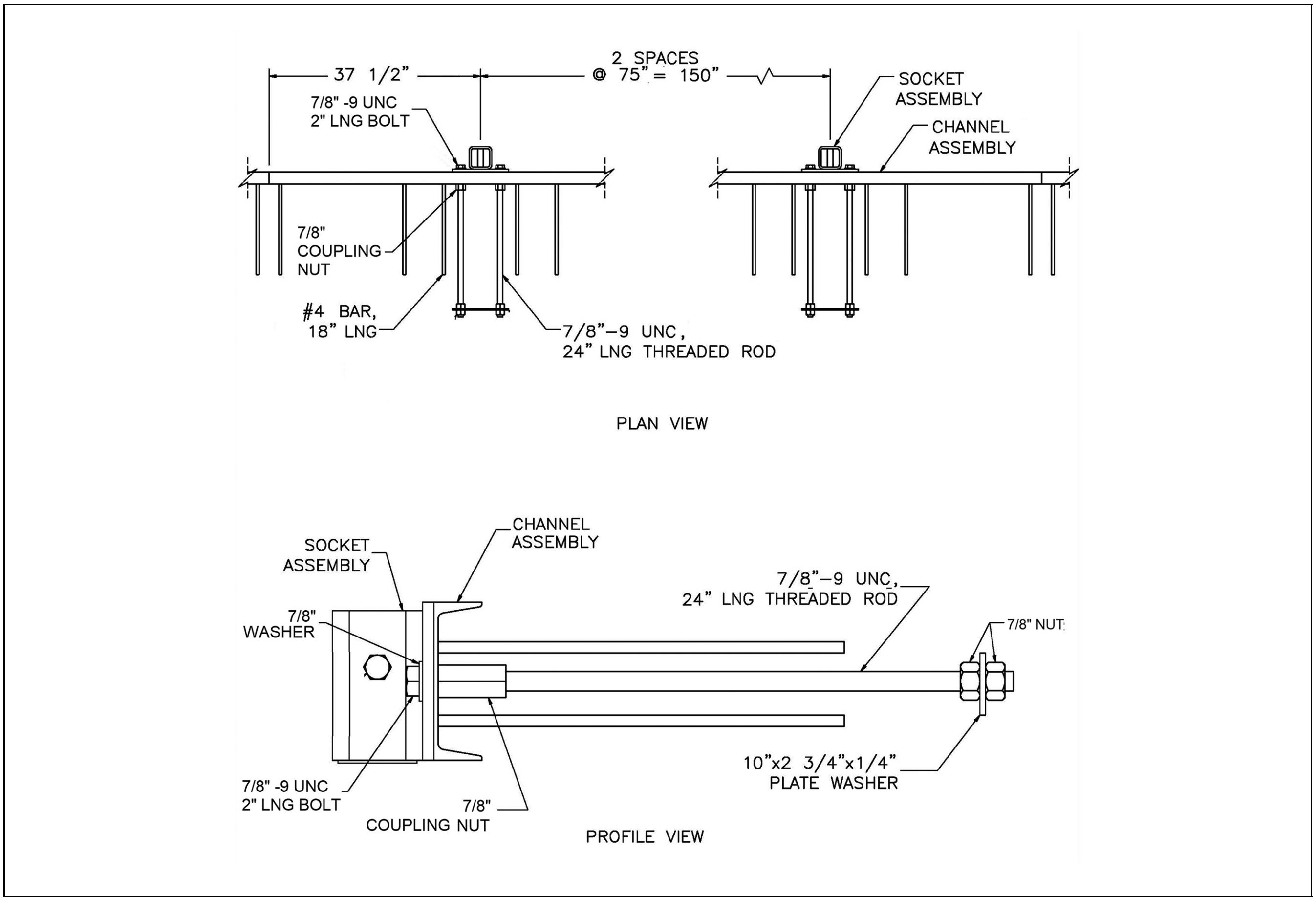

Figure 13. Post-to-deck attachment details.

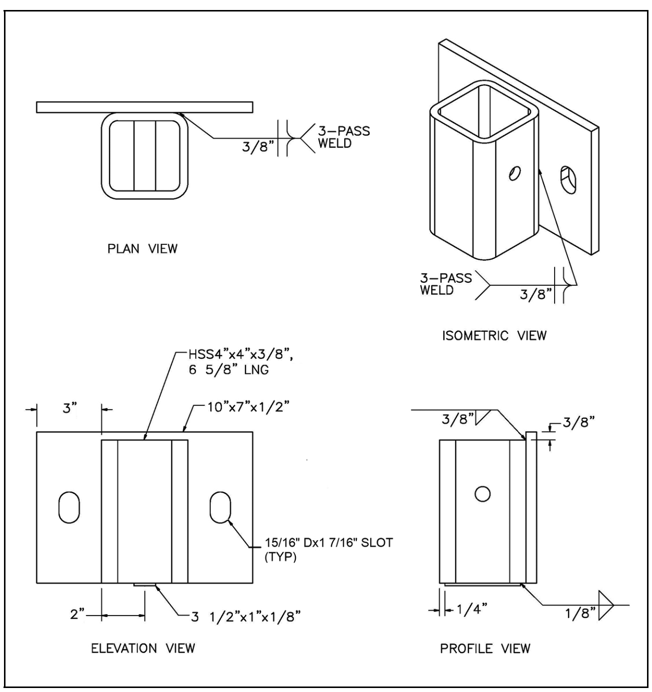

Figure 14. Socket assembly details.

deflections than observed during tests with the 1100C test vehicle. Therefore, MASH test 2-11 was deemed necessary to evaluate the post-to-deck connection strength of the new system, and MASH test 2-10 was determined to be non-critical. Should future knowledge gained from testing of this bridge rail or similar systems raise concerns about the new bridge railing's performance with small cars, it may become necessary to evaluate the bridge rail with the MASH 1100C vehicle.

MASH evaluation criteria are based on three appraisal areas: (1) structural adequacy; (2) occupant risk; and (3) vehicle trajectory after collision. Criteria for structural adequacy are intended to evaluate the ability of the bridge railing to contain and redirect impacting vehicles under controlled lateral deflections of the test article. Occupant risk evaluates the degree of hazard to occupants in the impacting vehicle. Post-impact vehicle trajectory is a measure of the potential of the vehicle to result in a secondary collision with other vehicles and/or fixed objects, thereby increasing the risk of injury to the occupants of the impacting vehicle and/or other vehicles. These evaluation criteria are defined in greater detail in MASH (*1*).

Figure 15. Test installation photographs.

Table 1. Crash Test Matrix for Longitudinal Barriers, *Manual for Assessing Safety Hardware* Test Level 2

Test article	Test designation no.	Test vehicle	Vehicle weight (lb)	Impact conditions	
				Speed (mph)	Angle (°)
Longitudinal barrier	2-10	1100C small car	2425	44	25
Longitudinal barrier	2-11	2270P pickup truck	5000	44	25

Full-Scale Crash Testing

Test no. N2BR-1 was conducted on the new TL-2 bridge railing in accordance with MASH test 2-11 test conditions. The 4999-lb pickup truck impacted the new TL-2 bridge rail at a speed of 44.2 mph and at an angle of 25.5°. The initial point of impact was 10 ft upstream from a rail splice in the middle of the bridge rail installation, which was picked to provide maximum loading to a rail splice within the system.

After initial impact, the new TL-2 bridge railing contained the pickup truck and smoothly redirected it back onto the bridge. As it was redirected, the vehicle's right-side tires extended over the edge of the deck, but the guardrail brought the vehicle back onto the simulated bridge deck in a stable manner with minimal roll. The right-front corner of the pickup truck extended a maximum of 38.4 in. over the edge of the bridge deck during the impact event. As the front of the vehicle was being brought back over the deck surface, the front-right tire snagged on the socket assembly supporting post no. 17, causing the wheel to disengage. The tire snag resulted in about a 10-*G* longitudinal acceleration pulse, which remained well below the MASH limits, and only minor pitch and roll displacements. After exiting the system, the brakes were remotely applied, and the vehicle veered back toward the system and impacted the MGS downstream of the bridge rail. The vehicle came to rest adjacent to the downstream anchorage 104 ft 5 in.

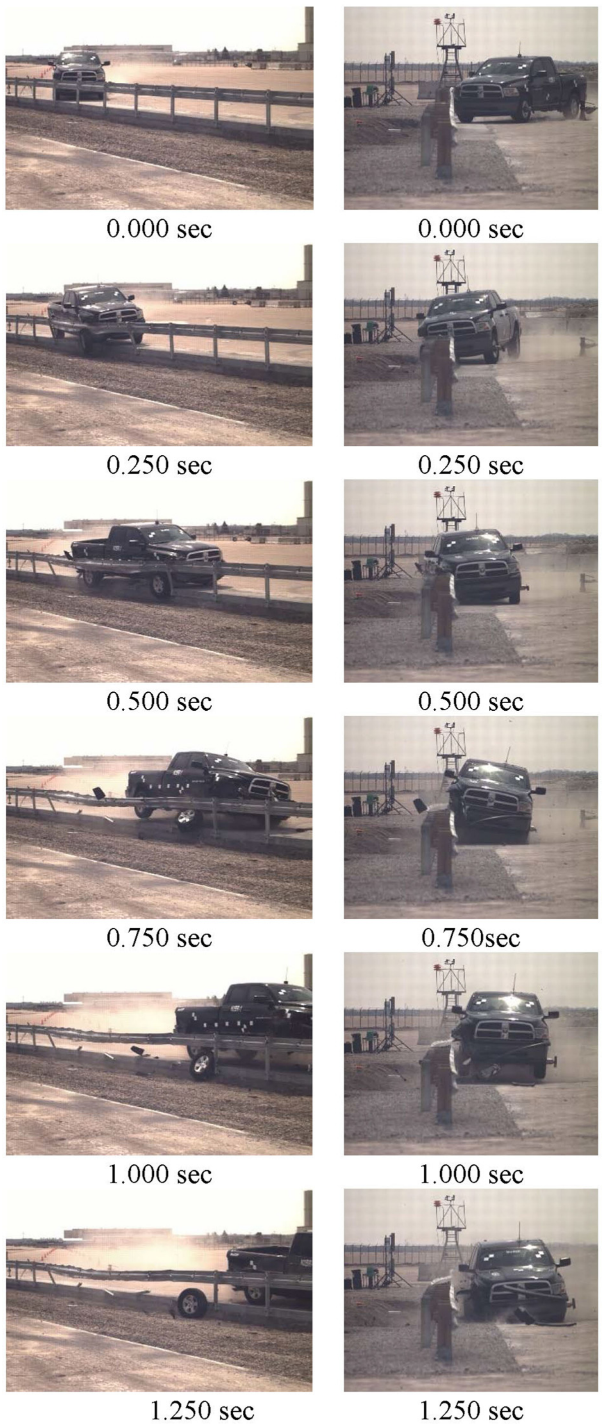

Figure 16. Sequential photographs, test no. N2BR-1.

downstream from the initial impact point. Sequential photos of the impact event are shown in Figure 16.

The length of vehicle contact along the barrier was approximately 27 ft. Damage to the bridge rail consisted of guardrail deformations and post bending within the contact region, as shown in Figure 17. Guardrail deformations consisted of rail flattening, kinking, and bending, and the guardrail was disengaged from five posts (a common and expected behavior). A total of six posts were plastically deformed. Two posts near initial impact were bent backward, whereas the next four downstream posts had been bent nearly 90° downstream as the vehicle traveled over them. Damage to the socket assemblies consisted of only minor scrapes and minimal deformations to their top edges. Four of the sockets had twisted downstream, but this rotation was the result of the slots in the mounting plates. All socket assemblies and attachment bolts could be reused. The maximum lateral dynamic deflection of the rail was 32.6 in. and the maximum permanent set of the railing was 20 in.

Table 2. Vehicle Accelerations and Angular Displacements, Test No. N2BR-1

Evaluation criteria	Test no. N2BR-1	MASH limits
Occupant impact velocity (ft/s)		
Longitudinal	−11.52	±40
Lateral	−11.55	±40
Occupant ridedown acceleration (G)		
Longitudinal	−10.98	±20.49
Lateral	5.74	±20.49
Maximum angular displacement (°)		
Roll	12.1	±75
Pitch	1.9	±75
Yaw	−32.3	not required

Damage to the vehicle was concentrated on the right-front corner of the vehicle. The right-front tire was disengaged, and the right-front bumper and fender were crushed inward. Minor dents and scrapes were observed along the right side and undercarriage of the vehicle. Deformations to the occupant compartment were minimal as they did not exceed 0.5 in.

On-board transducers were utilized to measure accelerations and angular displacements of the vehicle during the test. The calculated occupant impact velocities (OIVs) and maximum 0.010-sec average occupant ridedown accelerations (ORAs) in both the longitudinal and lateral directions are shown in Table 2. Note that the OIVs and ORAs were within the MASH limits.

Finally, the vehicle exited the barrier at an angle of −1.0°, and its trajectory did not violate the bounds of the exit box. Therefore, on review of all data presented here, test no. N2BR-1 was determined to be acceptable according to the safety performance criteria for MASH test 2-11. Further details can be found in the project report (*10*).

Adjacent Guardrail Length Considerations

For the new TL-2 bridge rail to function properly, additional guardrail and proper anchorage is needed on both

Figure 17. System damage, test no. N2BR-1.

the upstream and downstream ends of the bridge railing, similar to the as-tested configuration. Factors that should be considered to determine the minimum length of guardrail necessary for a complete system include: (1) the guardrail length of need required to shield the hazard, (2) terminal stroke length, (3) guardrail anchorage requirements, and (4) the minimum length needed to resist compression forces from crashworthy end terminals. When determining the minimum length of guardrail required adjacent to the bridge rail, all four factors should be considered. These factors are discussed independently in the following sections.

Length of Need Required to Shield Roadside Hazards

Roadside hazards within the clear zone require a certain length of guardrail upstream from the hazards to properly shield them from errant motorists. The *AASHTO Roadside Design Guide* provides equations for determining the length of guardrail necessary to shield hazards (*2*). These equations are dependent on site-specific variables, such as speed, runout length, clear zone, and the lateral extent of the hazard known as the area of concern. If the guardrail installation is not sufficient in length, the hazard is not truly shielded and still poses a risk to motorists.

Terminal Stroke Length

Terminal stroke length is defined as the maximum longitudinal vehicle stopping distance during head-on impacts into the end terminal. Sufficient stroke length is necessary to ensure proper end-terminal energy dissipation and that the vehicle comes to a stop before reaching the bridge, where it could roll off the edge of the deck. Terminal stroke length varies for each end-terminal system, so roadside engineers should refer to manufacturer specifications to determine the required stroke length for the end terminal intended for use within an installation. It is recommended that the TL-2 stroke length for the end terminal be used when evaluating system lengths for the new TL-2 bridge rail system.

Previously, 12.5 ft of standard guardrail has been recommended between a terminal and any MGS special applications, such as the new TL-2 bridge rail, to separate the different systems and ensure they do not negatively affect the performance of the other system (*11*). This 12.5 ft of separation guardrail has been recommended for both tangent and flared end terminals, as shown in Figure 18. It is recognized that the additional 12.5 ft of MGS is a conservative approach that may not be applicable or necessary in all cases, especially for very

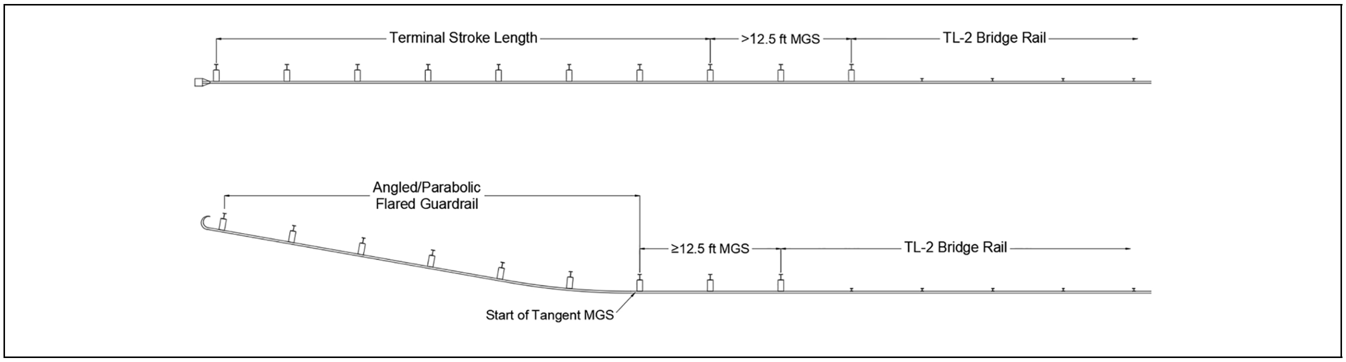

Figure 18. Separation of end-terminal stroke length and bridge rail.

low-volume roads where the risk of severe crashes is minimal and installation funds are limited.

Anchorage Requirements

For the TL-2 bridge rail and adjacent guardrail to function as intended, sufficient guardrail anchorage is required for the W-beam to develop the necessary tensile forces. Typical guardrail end anchorages consist of an anchor cable between the rail and the ground and two anchorage posts, among other components. Multiple crashworthy guardrail end anchorages and trailing end anchorages have been developed that provide adequate tensile capacity to the rail. One of these systems must be included on both ends of the bridge rail for proper system performance.

Most guardrail end terminals are considered to be gating systems, meaning that for impacts near the end of the terminal, the anchorage is expected to release and allow the vehicle to travel behind the installation. Thus, the beginning of the length of need for anchorages needs to be considered for the system to remain effective. For compression-based end terminals, the beginning of length of need is typically considered to be at post 3, whereas tension-based end terminals may begin length of need closer to post 1. Each end terminal is unique, so manufacturer's specifications should be followed for each terminal design. It is recommended to use a minimum of 12.5 ft of MGS between the beginning of length-of-need point on a terminal and the first bridge rail post to ensure proper performance of the system, as changing the posts in the contact region can change the length-of-need point and result in inadequate system performance.

Compression Terminal Force Resistance

Compression terminals require the guardrail to resist a certain amount of compressive forces as the vehicle is brought to a stop. After the guardrail anchorage is released at the beginning of an end-on impact, only the downstream support posts are left to provide resistance to the guardrail and prevent the entire installation (and vehicle) from translating downstream. Note, tension-based end terminals would not require downstream posts to resist impact loads, so this concern only applies to compression terminals.

The compression resistance applied to the guardrail by a post can be defined as the minimum between the post's longitudinal (weak-axis) bending strength, the post's torsional strength, and the shear capacity of the guardrail attachment bolt. These capacities were estimated for both standard W6x8.5 MGS posts and the S3x5.7 weak posts within the new bridge rail by using specified geometric and material properties for each component. Soil ground line and the tops of the sockets were assumed to act as a fixed end support for the posts when calculating the bending and torsional capacities of the posts.

The capacity of the S3x5.7 post was limited to 1.2 kips by the shear capacity of the $^{5}/_{16}$-in. diameter guardrail attachment bolt, whereas the strength of the W6x8.5 post was found to be 2.4 kips because of torsion failure with a 12-in. blockout. For shorter blockouts, the capacity of a W6x8.5 post would be limited by its weak-axis bending capacity of 2.8 kips. Further details can be found in the project report (*10*). Posts used within the end terminals on the downstream side of an installation would also resist the compressive forces in the W-beam. However, many terminal posts are weakened or breakaway posts, so these posts would require further analysis to determine their capacities.

The magnitude of the compressive forces applied to the guardrail varies by terminal because of the differences in energy absorbing mechanisms. Average compressive forces were previously determined through an analysis of full-scale crash testing and are shown in Table 3 (*12*). Peak end-terminal compressive forces have the potential to be greater than the average end-terminal forces. Should the designer wish to design for the case of peak end-terminal forces, a factor of safety may be utilized.

Table 3. End-Terminal Average Compressive Forces

End-terminal system	Average compressive force (kips)
BEST-350	18–22.5
ET-2000	12–21.3
ET-2000 plus	12–21.3
FLEAT-350	13.5–16.7
SKT-350	10.5–15.2
SKT-MGS	10.5
ET-Plus (27¾ in.)	15
ET-Plus (31 in.)	12.7
SGET	15.2
MSKT	12.6

Note: MGS = Midwest Guardrail System; BEST = Beam Eating Steel Terminal; ET = Extruder Terminal; FLEAT = Flared Energy Absorbing Terminal; SKT = Sequential Kinking Terminal; SGET = SPIG Guardrail End Terminal; MSKT = MASH SKT.

For the guardrail installation to resist the compression loads of the terminal, the following equation must be satisfied:

$$N_s P_s + N_w P_w > C \tag{1}$$

where N_S is the number of S3x5.7 bridge rail posts, P_S is the strength of an S3x5.7 post, N_w is the number of W6x8.5 MGS posts, P_w is the strength of a W6x8.5 post, and C is the compressive load for a given terminal. Values for the post strengths and terminal compression forces are known and were provided above; the number of bridge rail posts will be site specific. The only remaining variable is the number of MGS posts required to satisfy this equation, which can be used to then calculate the require MGS length by multiplying by 6.25 ft (post spacing). Note, the posts within the upstream terminal's stroke length should not be counted as part of N_w or the guardrail resistance, as these posts would be overrun by the impacting vehicle and disengage from the rail.

Conclusions and Recommendations

A new MASH TL-2 bridge rail was developed for use on rural, low-volume roads. The railing was based on previous weak-post, W-beam bridge rails but was modified to be a side-mounted system utilizing an innovative post-to-deck attachment design. The new bridge rail was then subjected to a combination of dynamic component tests and full-scale crash testing and found to be crashworthy to the safety performance criteria of MASH TL-2.

The new TL-2 bridge rail was developed using weak posts and W-beam guardrail, which resulted in a more cost-efficient design as compared with typical rural bridge rails. The railing is a completely side-mounted system that does not require any hardware on the deck surface. Thus, the traversable width of the bridge deck is maximized. The design also incorporates a socketed post design for quick and easy installation.

The simulated bridge deck and all of the socket assemblies remained undamaged following test no. N2BR-1. Additionally, none of the attachment bolts or coupling nuts were damaged. As such, repairs to the bridge rail would only include the removal and replacement of damaged W-beam segments and posts.

The minimum length of MGS installed adjacent to the guardrail was also investigated. Guidance was provided for determining the minimum system length, including factors for guardrail length of need to shield the hazard, terminal stroke length, guardrail anchorage requirements, and the installation length necessary to resist the terminal compression forces.

The new TL-2 bridge rail was designed to be compatible with both 7-in. thick CIP decks and 12-in. thick precast beam slabs. The 7-in. CIP deck was selected as the critical deck for full-scale crash testing and sustained no damage during the crash test. The new post-to-deck attachment hardware was evaluated via dynamic component testing when installed on a 12-in. thick precast beam-slab. These component tests also resulted in no damage to the deck or attachment hardware. Thus, the new TL-2 bridge rail is considered crashworthy in combination with both deck types.

The new TL-2 bridge railing developed here utilizes the same 31-in. tall W-beam and S3x5.7 weak posts as the two other MASH-crash-tested TL-3 bridge railings shown previously. Additionally, all three bridge rails perform the same way with post bending absorbing the impact energy while the deck and post-to-deck-attachment remain undamaged. The key difference in performance between the new TL-2 bridge rail and these other two MASH TL-3 railings is the post spacing for the TL-3 railings was reduced to 37.5 in. on-center. Therefore, if the post spacing of the new bridge railing developed here were reduced to 37.5 in. on-center, the system would be expected to perform similarly to the other systems and be crashworthy to MASH TL-3.

Finally, an analysis was conducted to evaluate the crashworthiness of the system at the connection between the railing and the adjacent MGS. This analysis was not included here because of limited space, but it is detailed in the project report (*10*). The analysis was conducted using computer simulations of both TL-2 and TL-3 impacts into the transition region and found that a 75-in. spacing between adjacent MGS strong posts and S3x5.7 bridge posts resulted in safe and smooth redirections. Thus, the MGS can be directly attached to the ends of the new bridge rail using a 75-in. spacing between the adjacent posts.

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Author Contributions

The authors confirm contribution to the paper as follows: study conception and design: S.K. Rosenbaugh, R.K. Faller, and R.W. Bielenberg; data collection: S.K. Rosenbaugh; analysis and interpretation of results: S.K. Rosenbaugh, R.K. Faller, and R.W. Bielenberg; draft manuscript preparation: S.K. Rosenbaugh and R.K. Faller. All authors reviewed the results and approved the final version of the manuscript.

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