Evaluation and Design of a TL3 Bridge Guardrail System Mounted to Steel Fascia Beams



Prepared by: Chuck A. Plaxico, Ph.D. T. Olaf Johnson, El

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The contents of this report reflect the views of the author(s) who is (are) responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of Ohio's Research Initiative for Locals, the Ohio Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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INTRODUCTION

The bridge superstructures on Ohio's local road system are often steel-stringer-beam designs supporting various non-concrete bridge deck types, including: timber, asphalt filled steel stay-in-place forms, and fiber reinforced composite. The bridge rail systems currently used on these bridges are typically side-mounted steel-post-and-beam designs that are mounted directly onto the steel fascia beam of the bridge superstructure. These designs have not been crash tested and are thus ineligible for use on Federal Aid projects. Further, there are no alternative side-mount systems currently available for mounting to the various bridge deck types used on the local road system.

The minimum requirement for federal-aid eligibility is that the system must meet Test Level 3 (TL3) crash performance criteria according to the National Cooperative Highway Research Program (NCHRP) Report 350 (R350) or its successor the AASHTO *Manual for Assessing Roadside Hardware* (MASH). The existing bridge rail designs that currently meet these standards involve mount designs that attach directly to the top or side of a reinforced concrete bridge deck; additionally, there are a few "eligible" bridge rail designs available for fastening onto timber decks.[*Mongiardini11*] The advantage of having a bridge rail that mounts directly to steel fascia beams is that it gives local bridge owners the option of building steel bridges with lower-cost bridge decks, while avoiding having to build excess deck width to accommodate a top-mounted bridge railing anchorage.

RESEARCH OBJECTIVES

The objectives of this project were to design a steel fascia beam post-mount for an existing TL3 bridge rail system, and to select an appropriate TL3 transition system for connecting the bridge rail to Ohio's w-beam guardrail systems. The bridge rail system selected for the baseline design was the side-mounted Illinois two-tube bridge rail. Detailed drawings for the baseline design are shown in Appendix A. This system is currently classified as a Report 350 Test Level 4 barrier and is thus eligible for use on federal-aid reimbursement projects.

On May 12, 2012, the FHWA instituted a new Federal-Aid Reimbursement Eligibility Process for roadside hardware including bridge railings that is to be used for requesting eligibility letters (i.e., formerly called acceptance letters). The new process details the evaluation procedures for new hardware designs as well as for evaluating modifications to currently accepted designs. When evaluating already accepted hardware, the design modifications are categorized in one of two classifications:

- Significant a change that has the potential to adversely affect the crash performance of the hardware. These types of changes require that new crash tests be performed.
- Non-Significant are changes that are relatively minor but it is not clear if the changes will adversely affect the safety performance. In these cases, a finite element analysis using LSDYNA can be used to demonstrate that the change does not adversely affect the crash performance of the hardware.

In this case the modifications to the baseline system were limited to the post-mount design; no changes were made to any part of the bridge rail design above the bridge deck surface. The

modified mount includes a structural steel tube 15 inches long with two 7/8" thick steel mounting plates welded to its ends. One end of the tube-mount is fastened to the steel fascia beam of the bridge superstructure using six 7/8" diameter A325 bolts; the other end of the tube-mount is fastened to a 1.0" thick mounting plate using two 1" diameter A325 bolts at the top of the mounting plate and two 5/8" diameter A325 bolts at the bottom of the mounting plate. The 1-inch thick mounting plate is welded to the flange of the W6x25 bridge rail post. The detailed drawings for the modified design are shown in Appendix B, and additional design development details are provided later in this report. The modified mount design was shown to provide stiffness equivalent to the original mount design, and the crash performance of the system with the modified mount was shown to be the same as that of the baseline system. The following sections of this report document the analysis procedures and results of the study.

GENERAL DESCRIPTION OF RESEARCH

Selection of Baseline Bridge Rail Design

A critical review of published literature and ongoing research was first conducted to investigate performance aspects of various side-mounted bridge rail systems that may be applicable to this project. The purpose of the review was to determine the most relevant bridge rail systems and mounting options to serve as candidate designs for further evaluation. There were three primary goals for the literature review:

- 1. To identify existing fascia mounted bridge rails currently eligible for use on Federal Aid projects (e.g., consider adoption of the system without any further analyses),
- 2. To identify existing side-mounted bridge rails currently eligible for use on Federal Aid projects that could readily be modified to accommodate attachment to steel fascia beams (e.g., consider modifying the mount and then seek acceptance under the original eligibility letter using the new Federal-Aid Reimbursement Eligibility Process),
- 3. Identify other side-mount designs *not* currently eligible for use on Federal Aid projects, but that have potential for successful performance under MASH TL3 impact conditions (e.g., consider modifying an existing system and then performing full-scale tests under MASH crash testing procedures).

The review was focused on existing side mounted bridge rail systems that meet FHWA eligibility requirements for use on Federal Aid projects, but also included older systems that were approved under NCHRP Report 230 MSL-2 or AASHTO PL1. The testing requirements for these two testing procedures are similar to those of NCHRP Report 350 TL3, with the primary differences involving vehicle type and/or impact speed for the large passenger test vehicle. Important information was, nevertheless, garnered from those tests regarding structural capacity of the bridge rail systems as well as their various post mount designs.

Based on the results from the literature review (see reference [*Plaxico15a*] for details) the Illinois Two-Tube Bridge Rail, which is a Report 350 TL-4 system, was selected as the baseline system for the project. A photo of the system is shown in Figure 1. This system has been full-scale crash tested and is thus eligible for modification and evaluation using finite element analysis under the new Federal-Aid Reimbursement Eligibility Process (FAREP).



Illinois Two-Tube (R350 TL4)

Figure 1. Candidate bridge rail selected as the baseline system for the project.

The Illinois Side Mount Bridge Rail consists of two structural tubes supported by W6x25 steel wide-flange posts spaced at 6'-3" on center. The posts are 4'-1¹/₄" long and are sidemounted to the concrete bridge deck using 5/8" diameter bolts. The upper and lower tubes are fabricated from 8"x4"x5/16" and 6"x4"x1/4" structural steel tube, respectively. The top of rail is 32 inches above the roadway surface. This system is included in the TF13 Bridge Rail Guide as SBB31d. [*TF13SBB31d*]

This system was full-scale crash tested at the Texas Transportation Institute (TTI) in 1993 under test conditions corresponding to the AASHTO 1989 GSBR PL2.[*Buth93; Buth97*] Test 7069-35 involved a 1,800-lb 1981 Honda Civic impacting the bridge rail at 59.9 mph and 20.1 degrees. Test 7069-36 involved a 5,400-lb 1985 Chevrolet pickup impacting the bridge rail at 60.4 mph and 20.4 degrees. The single unit truck (SUT) test (i.e., Test 7069-37) involved an 18,000-lb 1981 Ford SUT impacting the bridge rail at 51.4 mph and 14.7 degrees. The system successfully passed all three tests thus meeting the requirements for AASHTO Performance Level 2. The crash test reports for Test 7069-35, 7069-36, and 7069-37 are included in Appendices C, D and E, respectively.

The small car test resulted in no noticeable damage to the bridge railing or bridge deck. The pickup truck test resulted in 1-inch permanent deformation of the top rail and 0.75 inches deformation of the lower rail. The flanges on two posts were deformed as well as the angle stiffeners on the post-mount. The SUT test resulted in moderate damage to the railing. The upper and lower rails sustained gouges; the head of the lower bolt on the top rail at one of the posts was torn off; the angles at three of the post mounts were bent; the spacers at two post mounts were knocked loose and down; and there was 2.5 inches of deformation to the upper rail. This system was later given the NCHRP Report 350 equivalency rating of TL4 in the FHWA memorandum of May 30, 1997.[*FHWA97*]

Study Approach

The Illinois Two-Tube bridge rail was modified in this study to accommodate attachment of the system to steel bridge fascia beams. In order to conform to the classification of a "non-significant change" the modified mount design must provide a force-deflection response equivalent to the baseline post-mount to ensure that the performance of the bridge rail is unchanged for TL4 impact conditions. The basic approach for the study was to:

- 1) Determine the force-displacement response of the original post-mount using engineering calculations and physical testing,
- 2) Develop a modified mounting design(s) for attachment to steel bridge fascia beams,
- 3) Evaluate the force-deflection response of the modified post-mount(s) using FEA and physical testing,
- 4) Ensure that the loads transferred into the bridge superstructure in vehicle crashes do not result in excessive damage to the bridge superstructure,
- 5) Perform NCHRP Report 350 Test 4-12 simulation with impact conditions similar to the original full-scale test to verify that the modifications do not adversely affect crash performance, and
- 6) Perform MASH Test 3-10 and Test 3-11 impact simulations to assess crash performance of the system under the new crash testing procedures.

Steel bridge designs include various sizes of stringer beams, diaphragms and connection elements; for example, stringer beams on rural bridges may vary from as small as W12 structural sections to as large as W24. Thus, an important part of this study was to ensure that the bridge's structural elements (e.g., fascia beam, diaphragm and connection elements) are sized appropriately for the loads imparted during vehicular crashes. To further safeguard the bridge, a failure mechanism was incorporated into the post-mount design that would release the post from the post-mount during critical high-severity impact cases.

The analyses in the study were carried out using a combination of engineering calculations, pendulum testing and finite element analysis. The pendulum testing was performed at the Turner-Fairbank Highway Research Center's (TFHRC) Federal Outdoor Impact Laboratory (FOIL) in McLean, Virginia. The finite element analyses were carried out using LS-DYNA, which is a non-linear, dynamic, explicit finite element code that is very efficient for the analysis of vehicular impact and is used extensively by automotive industries, as well as roadside safety design engineers, to analyze crashworthiness of vehicles and hardware designs.[LSDYNA15]

Since the scope of work relied heavily upon finite element analysis, it was therefore necessary to validate the model(s) to gain confidence in their results. Detailed finite element models for the original post-mount design as well as several modified post-mount designs were developed and used to simulate impact of a rigid pendulum striking the post at 15 mph. Each of these simulated impact cases were tested at the Federal Outdoor Impact Laboratory (FOIL) and compared with results of the analyses. Note that the validation of the complete baseline bridge rail model and test vehicle was not performed for this engineering analysis approach; however,

the Test 4-12 analysis of the final modified design was compared to the baseline test to confirm engineering analysis results.

A summary of the validation process is presented in the following section. Additional details regarding development, validation and application of the finite element analysis (FEA) models, as well as the details regarding the testing procedures will be included as part of the discussion presented in the "Results" section of this report.

Quantitative Validation Procedure

As part of the validation process, the validation procedures presented in NCHRP Web Document 179 (W179) were used to assess the fidelity of the model. [*Ray11*] The validation procedure has three steps:

- 1. Solution verification: Indicates whether the analysis solution produced numerically stable results (ensures that basic physical laws are upheld in the model).
- 2. Time-history evaluation: Quantitative measure of the level of agreement of time-history data (e.g., x, y, z accelerations and roll, pitch, and yaw rates) between the analysis and test.
- 3. Phenomena Importance Ranking Table (PIRT): A table that documents the types of phenomena that a numerical model is intended to replicate and verifies that the model produces results consistent with its intended use.

Time-History Evaluation

The RSVVP (Roadside Safety Verification and Validation Program) software, which was developed as part of NCHRP Project 22-24, computes the comparison metrics between analysis results and full-scale test data. RSVVP computes fifteen different metrics that quantify the differences between a pair of curves. Since many of the metrics share similar formulations, their results are often identical or very similar. Because of this, it is not necessary to include all of the variations. The metrics recommended in Report W179 for comparing time-history traces from full-scale crash tests and/or simulations of crash tests are the Sprague & Geers metrics and the ANOVA metrics. The Sprague-Geers metrics assess the magnitude and phase of two curves while the ANOVA examines the differences of residual errors between them. The definitions of these metrics are shown below:

Sprague-Geers:

Magnitude
$$(M) = \sqrt{\frac{\Sigma c_i^2}{\Sigma m_i^2} - 1}$$

Phase $(P) = \frac{1}{\pi} \cos^{-1} \frac{\Sigma c_i m_i}{\sqrt{\Sigma c_i^2 \Sigma m_i^2}}$
Comprehensive $(C) = \sqrt{M^2 + P^2}$

ANOVA:

Residual Error
$$(\overline{e}^r) = \frac{\Sigma(m_i - c_i)}{m_{\max}} \cdot \frac{1}{n}$$

Standard Deviation $(\sigma) = \sqrt{\frac{1}{n}\Sigma(m_i - c_i - \overline{e}^r)^2}$

Where,

 $c_i = calculated quantities$

 m_i = measured quanties

 $m_{\rm max} =$ maximum measured value

 \overline{e}^{r} = relative average residual error

 σ = relative standard deviation

n = number of data points

Time-History Evaluation Acceptance Criteria

Once a measure of comparison is obtained using a quantitative metric, it is necessary to establish an acceptance criterion for deciding if the comparison is acceptable. Because of the highly nonlinear nature of crash events, there are often considerable differences in the results of essentially identical full-scale crash tests – this was demonstrated in the W179 report. Likewise, a computational model may not match "exactly" the results of a physical test, but the difference should be no greater than what is expected between physical tests. The approach taken in the W179 was to determine the realistic variation in the deterministic shape comparison metrics for a set of identical physical experiments and use that variation as an acceptance criterion. The current acceptance criteria is based on the results of a quantitative comparison of ten essentially identical full-scale crash tests that were performed as part of the ROBUST project involving small car impact into a vertical rigid wall at 100 km/hr and 25 degrees.[*ROBUST02; Ray11*] The resulting acceptance criteria recommended by W179 for assessing the similarity of two time-history curves are:

- Sprague-Geers
 - Magnitude should be less than 40 percent
 - Phase should be less than 40 percent
- ANOVA metrics
 - Mean residual error should be less than 5 percent
 - Standard deviation should be less than 35 percent.

Phenomena Importance Ranking Tables (PIRT)

The PIRT includes evaluation criteria corresponding to NCHRP Report 350 for TL-3 impacts and is patterned after the full-scale crash test evaluation criteria in NCHRP Report 350.[*Ross93*] The values for the individual metrics from the full-scale test and the computer analysis were reported and both the relative difference and absolute difference for each phenomenon were computed. If the relative difference is less than 20 percent or if the absolute difference is less than 20 percent of the acceptance limit in NCHRP Report 350, then the phenomena are considered to be replicated.

Typical Steel Bridge Structure Details

US Bridge provided construction drawings for several of their previous bridge projects. This information was used as guidance in selecting the bridge structure components to be used in the analyses. In particular, those components considered critical in carrying the loads imposed on the bridge structure through the mount include the fascia beam, the diaphragm, the diaphragm-to-fascia structural connector, and to a lesser extent the bridge deck. A summary of these components as defined in the construction drawings are shown in Table 1.

	Stringer Beam				Diaphragm			Bolt	Deck	Mount	Stringer	Deck
Drawing		Span			Length	Connector	Connector			Length	Spacing	Width
Number	Size	(ft)	Material	Size	(ft)	(fascia)	(stringer)	Size	Туре	(in)	(in)	(ft)
51694	W12x58	24	A709 Gr 50	C8x11.5	3.4	WT5x22.5	L5x3.5x3/8"	7/8"	3"x9" 5 ga. Corrugated Steel	20.4	22.3	24
52943	W12x58	28	A709 Gr 50	C8x11.5	3.6	WT5x22.5	L5x3.5x3/8"	7/8"	3"x9" 5 ga. Corrugated Steel	26.2	19.8	22
52455	W14x30	12	A709 Gr 50	C8x11.5	3.6	WT5x22.5	L5x3.5x3/8"	7/8"	5-3/16" Open Grid Decking	18.0	15.7	17
52784	W14x48	18	A709 Gr 50	C10x15.5	3.2	WT5x22.5	L5x3.5x3/8"	7/8"	3x6 Timber	26.4	14.0	16
52805	W16x36	12	A709 Gr 50	C10x15.3	5.2	WT5x22.5	L5x3.5x3/8"	7/8"	5-3/16" Open Grid Decking	17.7	22.0	23
52092	W16x77	28	A709 Gr 50	C12x20.7	6.3			3/4"	7" Precast Concrete	30.0	19.5	22
53021	W18x65	20	A709 Gr 50	C10x15.3	2.8	WT5x22.5	L5x3.5x3/8"	7/8"	3x6 Nail Laminated Timber	34.0	22.2	25
52055	W21x62	30	A709 Gr 50	C12x30	3.5	WT5x22.5	L5x3.5x3/8"	7/8"	3"x9" 5 ga. Corrugated Steel	18.4	26.5	28
52966	W24x94	47	A709 Gr 50	C12x30	4.1	WT5x22.5	L5x3.5x3/8"	7/8"	3"x9" 5 ga. Corrugated Steel	24.3	22.0	24

Table 1. Typical components used in steel bridge superstructur
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RESULTS

AASHTO LRFD Strength Calculations for Baseline System

The original design for the Illinois Two-Tube bridge rail design was evaluated according to the strength calculations of Appendix A13 of the AASHTO LRFD Bridge Design Specifications 2012 [*AASHTO12*]. The calculations, which are shown in Appendix F, were carried out based on TL4 loading conditions considering only lateral loads. From Table A13.2-1 of the AASHTO LRFD Bridge Design Specifications, the design lateral load for the bridge railing is 54 kips. The load is to be applied at 34 inches above the bridge deck surface, based on the effective height of the vehicle rollover force as calculated from Equation A13.2-1. The shear resistance of the W6x25 post for these load conditions was calculated to be 20 kips which corresponds to a resultant moment of 680 kip-in. The ultimate resistance for the baseline system was calculated to be 58 kips which meets the TL4 requirements (i.e., >54 kips).

These same calculations were also applied to the Illinois Two-Tube bridge rail design with the post mounted 10 inches below deck surface (e.g., consistent with typical mounting location onto fascia beam) – see Appendix G for details. The mounting connection was assumed to be rigid in these calculations. The shear resistance for the W6x25 post for this case was calculated to be 15.5 kips and the total lateral load capacity was computed to be 54 kips, which also meets TL4 requirements. However, in order to achieve the same shear resistance of the original post design (i.e., 20 kips), the minimum required plastic modulus for the post would be 24.5 in³. Since changing the posts' sectional dimensions (i.e., W6x25) is not permitted in the FAREP requirements regarding system modifications, the target shear and moment resistance of the post was achieved by stiffening the post and mount below the deck surface.

Development of Fascia Mount Design

Although it does not occur frequently, it is possible that the bridge rail will experience impact severities greater than TL4 at some point in time. It is therefore important to ensure that the strength of the system is adequate for TL4 loading, but also that the loads transferred into the bridge structure during loading, particularly those of higher impact severities, do not result in excessive damage to the bridge superstructure.

From the LRFD calculations shown previously, the resistance of the post under TL4 loading conditions resulted in a lateral force of 20 kips applied at 34 inches above grade, which corresponds to a resultant moment of 680 kip-inches on the mount. Since the post is only 32 inches tall, the loading point had to be lowered in order for the striker to make physical contact with the post; the load point was arbitrarily set to 25.75 inches above grade (i.e., 28.75 inches above the top mounting bolt). To obtain a moment equal to 680 kip-inches (i.e., TL4 condition), the required load applied under these conditions is 23.7 kips.

Physical testing was performed to more accurately determine the lateral load response for the original post-mount system. The test involved a 2,372-lb rigid pendulum impacting the post at 25.75 inches above grade at 15 mph. More details of the test setup and conduct are presented later in the *Strength Assessment of Proposed Design Options via Pendulum Tests* section of this report. The resulting post response is shown in Figure 2. The LRFD calculated load of 23.7 kips occurred at approximately 4-inches deflection of the post, while the peak response at 8.7-inches deflection was 28.6 kips. Recall that the permanent deflection of the post in the full-scale test was measured as 2.5 inches. The dynamic deflection was approximated from the crash test video to be between 4 and 6 inches. Thus, the lower bound for the TL4 load condition was taken as 23.7 kip at 4-inches post deflection, and the upper bound was taken as 27 kips at 6-inches post deflection.



Figure 2. Load response of the original post design with load applied at 28.75 inches above grade.

Mount Design

The basic mount design concept consisted of a structural steel tube with mounting plates welded to each end, as illustrated in Figure 3. The width for the tube section was set to 6 inches, which was equal to the width of the flange of the W6x25 steel post.



Figure 3. Basic mount design concept.

The deck thickness was taken to be 8 inches which, when considering the flange thickness of the fascia beam and allowing for a reasonable gap between the top of the mount tube and the fascia flange, results in the top of the mount being located approximately 10 inches below the deck surface. For the upper bound TL4 loading condition of 27 kips applied at 25.75 inches above grade, the minimum plastic section modulus for the tubular mount was calculated to be 25.7 in³, as shown in Figure 4. The selected mounting-tube sizes and their corresponding section moduli are also shown in Figure 4.

- Tube height, h = 14 in
- P = 27 kip (Max TL4)
- F_y = 46 ksi
- M_p = 27*43.75 =1,181 kip-in
- $Z_{min} = M_p / F_y = 25.7 \text{ in}^3$

Selected Sections:

Tube Section	Z _x (in³)	Z _y (in³)	A (in²)								
12 x 6 x ¼	33.2	20.6	8.59								
14 x 6 x ¼	42.3	23.4	9.59								





Post Stiffener Design

The increased length of the modified post resulted in an increased moment on the post at the mount connection. To compensate for this, stiffening plates were positioned between the flanges of the post and welded in place. Two different designs were selected and optimized for application with the 14" tall tubular mount:

- Design A: Consisted of two plates 8.5 inches long and ¹/₄" thick, oriented vertically and positioned at the edges of the post flange. The top of the plate was positioned 5 inches above the top of the tube mount. The total area of the ¹/₄" plates used for the stiffeners was 98 in². The total weld length was 34 inches.
- Design B: Consisted of four plates ¹/₄" thick, two on each side, fitted horizontally between the post's flanges. The top plates were positioned at 5 inches above the top of the tube mount and the lower plates were positioned at ¹/₄ inch above the top of the tube mount. The total area of the ¹/₄" steel plates used for the stiffeners was 62.2 in². The total weld length was 45 inches.

In both cases, a 1" thick plate was welded onto the front-side of the post to enable connection to the mount structure. The two designs are shown in Figure 5 below.





Strength Assessment of Proposed Design Options via Pendulum Tests

Dynamic pendulum impact tests were performed to measure the force-deflection response of select post-mount designs. The purpose was to: (1) compare the response of the new designs to the original baseline design to ensure that the new designs have equivalent stiffness compared to the original design and (2) to provide data for validation of the finite element models of the post and mount systems. The impact tests were conducted at the Federal Outdoor Impact Laboratory

(FOIL) at the Turner-Fairbank Highway Research Center located in McLean, Virginia. The physical testing performed in this task involved a rigid pendulum impacting against the post-mount designs where the tubular mount was bolted onto a relatively stiff structure to create a "quasi-fixed" boundary condition.

The test matrix included seven post mount designs, including the original post-mount design of the Illinois Two-Tube Bridge Rail system. The tests were set up to achieve a target displacement of approximately 9 inches. The intent was to apply greater loading on the posts than that measured in the Report 350 TL4 tests but not so excessive that the pendulum could not be stopped by the post-mount system.

Test Article Designs

The post-mount designs selected for the test program are shown in **Figure 6** and were chosen based on preliminary analyses that indicated that the force-deflection response was approximately equivalent to that of the baseline post-mount design. The details of the preliminary analyses can be found in Plaxico *et al.*[*Plaxico15b*] The designs included two crosssection sizes for the mounting tube, i.e., HSS 14"x6"x ¼" and HSS 12"x6"x ¼". The 14 inch tall tube was expected to be sufficient for mounting to fascia beam stringers composed of W16x40 steel structural sections and larger, while the 12 inch tall tube would be used for mounting onto steel fascia beams composed of W14x30 and larger W14 sections.

The designs also included two different post stiffener designs, labeled as Post Stiffener A and Post Stiffener B in Figure 6. Photos of these two stiffer designs are also shown in Figure 7. Post Stiffener A consisted of two plates 8.5 inches long and ¹/₄" thick, oriented vertically and positioned at the edges of the post flange. The top of the plate was positioned 5 inches above the top of the tube mount. Post Stiffener Design B consisted of four plates ¹/₄" thick, two on each side, fitted horizontally between the post's flanges. The top plates were positioned at 5 inches above the top of the tube mount and the lower plates were positioned at ¹/₄ inch above the top of the tube mount. In both cases, a 1" thick plate was welded onto the front-side of the post to enable connection to the mount structure. The weld between the post mounting plate and the tube mounting plate and post stiffeners to post) were 5/16" fillets.

The designs also included two different lengths for the HSS tube mounts: 15 inches and 21 inches. The materials (including hardware), fabrication, and shipping of the test articles were provided to the project by U.S. Bridge at no cost to the project. The detailed shop drawings for the test articles are provided in Appendix H. The test matrix for the study is shown in Table 2.





Figure 6. Drawings for the seven post-mount designs tested.

Figure 7. Photos of (a) Post Mount Design A and (b) Post Mount Design B.

	Test Articles		Post				Mount			Mour	ting Bol	Impact Conditions		
Design	Case	Test No.	Size	Length (in)	Post Stiffener	Mount Plate thk (in)	Tube Size (in)	Length (in)	Plate Thick (in)	Туре	Top dia. (in)	Bottom dia. (in)	Pendulum Weight (lb)	Impact Speed (mph)
1	Baseline	15009A	W6x25	51	-	-	-	-	-	A325-1	1	5/8	2,372	15.0
2	14A-15m	15009G	W6x25	56	А	1	14x6x 0.25	15	0.75	A325-1	7/8	3/4	2,372	15.9
3	14B-15m	150091	W6x25	56	В	1	14x6x 0.25	15	0.75	A325-1	7/8	3/4	2,372	14.1
4	12A-15m	15009J	W6x25	54	A	1	12x6x 0.25	15	0.75	A325-1	7/8	3/4	2,372	15.1
5	12B-15m	15009K	W6x25	54	В	1	12x6x 0.25	15	0.75	A325-1	7/8	3/4	2,372	14.9
6	14B-21m	15009F	W6x25	56	В	1	14x6x 0.25	21	0.75	A325-1	7/8	3/4	2,372	15.2
7	12A-21m	15009H	W6x25	54	А	1	12x6x 0.25	21	0.75	A325-1	7/8	3/4	2,372	14.5

Table 2. Test matrix for Test Series 15008 – lateral impact into scaled pier designs.

A - Vertical stiffener plates

B - Horizontal stiffener plates

Test Setup

A schematic of the test setup is shown in Figure 8 and a photo of the setup is shown in Figure 9. The test fixture entails a 1" thick steel plate that covers the complete front face of the pit area. The material immediately behind the pit wall was composed of soil (properties unknown). A 2"-thick steel plate was laid on top of the ground approaching the pit. This plate was bolted to a concrete foundation at one end and welded to the top of the 1" thick wall-plate. At the location of the post-mounts, a two-inch thick plate was welded onto the side of the wall-plate. The 2" plate and the 1" wall plate were then drilled and tapped for receiving the mounting bolts. To reduce the lateral deflections of the wall-plate at the lower edge of the mount during impact, two 45 inches long steel tubes with cross-section 2"x4"x1/4" were oriented horizontally and placed just below the 2" thick plate and then welded to the wall-plate.

Impact Conditions

The pendulum struck the face of the post flange at a nominal speed of 15 mph. The weight of the pendulum was 2,372 lb and the impact point for all cases corresponded to a height of 25.75 inches above the bridge surface. For the baseline case, in which the post would be mounted directly to the side of a concrete bridge deck, the corresponding impact point relative to the centerline of the top mounting bolt was 28.75 inches, as illustrated in Figure 10. For all other cases, in which the post would be mounted onto the side of the bridge fascia beam, the corresponding impact point relative to the centerline of the top mounting bolt was 35.75 inches.



Figure 8. Schematic of the test setup.



Figure 9. Photo of the test setup.



Figure 10. Sketch of impact locations relative to the centerline of the top mounting bolts.

Equipment and Instrumentation

Pendulum Device

A 2,372-lb rigid pendulum was used in the tests. The pendulum impact head included a semirigid nose made of wood and covered with 12-gauge sheet steel, as shown in Figure 11.



Figure 11. 2,372-lb pendulum device with semi-rigid nose.

Accelerometers

Impact forces were back calculated from accelerometer data. The accelerometers were mounted onto the back of the pendulum mass to measure acceleration of the striker during impact. Two accelerometers were used to record data in the x-direction (forward direction) and the third recorded data in the z-direction (vertical direction). Figure 12 provides a schematic showing the locations of the accelerometers.

ACCELEROMETERS LOCATIONS AND CHANNEL ID



Pendulum top view witccelerometer blocks

СН.	LOCATION X (cm) From frt. Y (cm) From lft frt. corner		Z (cm) From ground	SERIAL NO.	AXIS	
1	Rear Center of Pend.	43.0	127.0	71.0	D12748	-X
2	Rear Center of Pend.	43.0	127.0	71.0	D12150	-X
3	Rear Center of Pend.	43.0	127.0	71.0	D12820	-Z

Figure 12. Schematic of the accelerometer instrumentation for the pendulum tests.

The x-channel accelerometer data located on the back of the pendulum was processed to obtain various response measures from the impact tests. The acceleration data was filtered using an SAE Class 60 filter with a cutoff frequency of 100 Hz. The impact force-time history response was approximated by multiplying the acceleration-time history curves by the total mass of the pendulum. The acceleration data was then integrated to obtain velocity-time history, and again integrated to obtain the displacement-time history of the pendulum. This information was used to generate force-deflection curves for the impact.

Photography Cameras

The tests were also recorded using five high-speed cameras with an operating speed of 500 frames per second and two digital video cameras (\sim 60 fps). The camera specifications and locations are shown in Figure 13. The accelerometers and the high-speed video were triggered using pressure tape switches when the pendulum contacted the test specimens. The test setup and results were also documented with pre- and post-test photographs.

NO.	CAMERA	LENS	LENS (MM)	ZOOM	RESOLUTION (PIXELS)	SPEED (FPS)	LOCATION
1	GX-1	Nikon	18-35	35	1280 X 1024	500	Right Perp
2	GX-1	Nikon	85	85	1280 X 1024	500	Left Perp
3	GX-1	Nikon	24-85	70	1280 X 1024	500	Left Rear <u>Iso</u>
4	GX-1	Nikon	24-85	24	1280 X1024	500	Right Rear <u>Iso</u>
5	K3R	Nikon	85	85	1280 X 1024	500	Rear Perp



Figure 13. Locations for the high-speed cameras used in Test Series 15008.

Test Results

A summary of the test results is shown in Table 3 which includes peak force, peak displacement and/or displacement at failure of the post-mount system. For several of the tests, an inertial force spike occurred upon impact with the post (refer to force history plots in the following sections). This spike was associated with the mass and elastic properties of the pendulum, as well as those of the pendulum head and the post (e.g., stress-wave phenomena) rather than to the bending resistance of the post. Thus, this initial spike was not considered when determining the maximum force values. A summary sheet for each test is shown in Appendix I and sequential views for the tests are shown in Appendix J.

			Impact Co	onditions	Results					
Те	est Articles	s	P	ost	Mou	nt	Pendulum	Impact	Peak	Peak
			Length	Post	Tube Size	Length	Weight	Speed	Force	disp.
Design	Case	Test No.	(in)	Stiffener	(in)	(in)	(lb)	(mph)	(kips)	(in)
1	Baseline	15009N	51	-			2,372	15.0	28.2	8.8
2	14A-15m	15009G	56	А	14x6x25	15	2,372	15.9	30.9	12*
3	14B-15m	150091	56	В	14x6x25	15	2,372	14.1	30.2	9.2
4	12A-15m	15009J	54	А	12x6x25	15	2,372	15.1	28.3	9.4
5	12B-15m	15009K	54	В	12x6x25	15	2,372	14.9	27.8	9.7
6	14B-21m	15009F	56	В	14x6x25	21	2,372	15.2	30.6	8.8
7	12A-21m	15009H	54	А	12x6x25	21	2,372	14.5	28.1	9.3

Table 3. Summary of results for Test Series 15009.

A - Vertical stiffener plates

B - Horizontal stiffener plates

* The weld between the post mounting plate and the tube mounting plate fractured during the test.

Figure 14 and Figure 15 show the force-deflection curves and force-time history curves for the test specimens involving the HSS 14"x6"x1/4" and the HSS 12"x6"x1/4" tube-mounts, respectively, compared to the baseline post-mount response. Excluding the initial inertial spike, the maximum force for the modified tube-mount designs ranged from 27.8 to 30.9 kips, and the peak displacements ranged from 8.8 inches to 9.7 inches. These results compared well to the baseline test which had a peak force of 28.7 kips and peak displacement of 8.8 inches, as illustrated in Figure 16 and Figure 17. In Test 15009G (for Case 14A-15m) the weld broke just as the pendulum was coming to a stop (which indicated that the weld strength was just below the critical strength). That case ultimately resulted in a maximum deflection of 12.0 inches. Figure 18 and Figure 19 show images from the highspeed videos taken at the time of maximum post deflection for each case, including the baseline case.



Figure 14. Force vs. displacement and force-time history response for test specimens involving the HSS 14"x6"x1/4" tube mounts with modified weld compared with baseline response.



Figure 15. Force vs. displacement and force-time history response for test specimens involving the HSS 12"x6"x1/4" tube mounts with modified weld compared with baseline response.



Figure 16. Bar graph showing (a) peak force and (b) peak displacement results for the 14"x6"x1/4" mount test cases.



Figure 17. Bar graph showing (a) peak force and (b) peak displacement results for the 12"x6"x1/4" mount test cases.



Figure 18. Images from the high-speed videos at the point of maximum displacement and force for (a) Baseline Case, (b) Case 14A-15m, (c) Case 14B-15m and (d) Case 14B-21m.



- (d)
- Figure 19. Images from the high-speed videos at the point of maximum displacement and force for (a) Baseline Case, (b) Case 12A-15m, (c) Case 12B-15m and (d) Case 12A-21m.

Conclusions Regarding Pendulum Tests Results

For the 14"x6"x ¹/₄" mounting tube, all cases (e.g., Stiffener A and B, tube length 15" and 21") resulted in equivalent or slightly higher peak force values than the baseline post-mount system with essentially equivalent overall force-deflection response. Similarly, for the 12"x6"x ¹/₄" mounting tube, all cases resulted in essentially equivalent peak force compared to the baseline system, as well as resulting in essentially equivalent overall force-deflection response. In all cases, the tube-mounts were fastened to a semi-rigid fixture. Thus, the stiffness response will be somewhat reduced when fastened to the steel fascia beam.

Finite Element Model Development and Validation of Post-Mount

The pendulum impact tests conducted in Test Series 15009 were used to validate the finite element models. Detailed finite element models of the seven designs were developed and the finite element analysis code LS-DYNA was used to simulate the tests. The components of these post-mount designs are similar to those that were modeled and validated in previous research projects [*Plaxico02; Plaxico07; Plaxico13*]; therefore, the modeling methodology and material model characterization used in those studies were implemented in this study where appropriate.

Figure 20 shows various views of a typical finite element model that was used in the study. The cross-section of the post was modeled according to the dimensional specifications for W6x25 structural steel (i.e., depth = 6.38 in, flange width = 6.08 in, web thickness = 0.32 in, and flange thickness = 0.455 in). The material properties were characterized based on ASTM A709 Grade 50 steel. The post was modeled with thin-shell Belytschko-Tsay (BT) elements (Type 2 in LS-DYNA) with five integration points through the thickness. The part was meshed with a nominal element size of 0.5×0.5 inches (12.7×12.7 mm).

The welded connection of the posts to the 1-inch thick mounting plate was modeled using the *constrained_general_weld_spot option in LS-DYNA. An example of the weld-constraint connections between the top of the mounting plate and the post flange is shown in the left most image of Figure 20. The forces in the weld connections were collected and monitored during the analyses which will be used to determine the appropriate design strength for the welds. The 1-inch thick base plate was modeled with three layers of thick-shell elements with nominal size 0.5 x 0.75 x 0.33 inches with five integration points through the thickness of each element. The boltholes in the mounting plate were modeled with diameter of 1 inch and the elements around the immediate surface of the holes were meshed with nominal size of 0.35 x 0.35 inches (9x9 mm).

Two different tubular sections were used in the designs: HSS 14"x6"x1/4" and HSS 12"x6"x1/4". The tubular sections were modeled according to their specified dimensions with material conforming to ASTM A500 Grade B. The minimum yield and tensile strength for this material is 46 ksi and 58 ksi, respectively. The structural tubes were modeled with thin-shell BT elements with nominal size of 0.5 x 0.5 inches.

The welded connection of the tubes to the 3/4-inch thick mounting plate was modeled using the *contact_tied option in LS-DYNA which creates an automatic constraint between the nodes of the tube and the mounting plate. This connection assumes that these welds do not fail during impact as was verified in the physical tests. The 3/4-inch thick base plate was modeled with two layers of thick-shell elements with nominal size $0.7 \times 0.7 \times 0.3/8$ inches with five integration points through the thickness of each element. The bolt-holes for this mounting plate were also modeled with diameter of 1 inch with nominal mesh size of 0.35×0.35 inches (9x9 mm).

The bolts fastening the post-mount plate to the tube-mount plate were modeled explicitly using beam elements to model the bolt shaft and rigid solid elements to model the bolt head and nut. The 0.24-inch thick washers were modeled using thick shell elements with rigid material properties. Contact between the bolts and the mount components were defined in LS-DYNA using the "contact_automatic_general" option. The bolts were pre-tightened in the analysis prior to impact loading to ensure that no gaps were present between the contacting components.



Figure 20. Representative finite element model used to simulate Test Series 15009.

Model Setup and Analysis Cases

A photo of the setup and the FEA model is shown in Figure 21. The back wall-plate was modeled with fixed boundary conditions around the edges of the plate. Two 45-inches long horizontal steel tubes with cross-section 2"x4"x1/4" were placed just below post-mount with spot-weld constraints simulating the welds between the tube and wall.



Figure 21. Photo of the test setup and FEA model.

The pendulum mass was modeled with solid elements with elastic material with Young's modulus of 29,000 ksi and Poisson's ratio of 0.3. The pendulum head was modeled with solid elements with wood properties characterized using *Mat_Wood_Pine in LS-DYNA with default properties for DS-65 (i.e., $Qual_T = -1$ in LS-DYNA).

The pendulum struck the post at 90-degrees on the face of the post flange. The impact speed was set as 15 mph for all cases (note that the impact speed of the pendulum in the physical tests varied slightly from test to test). The weight of the pendulum was 2,372 lb and the impact point for all cases corresponded to a height of 25.75 inches above the bridge surface, consistent with the physical tests. Refer to the previous chapter for more details on the impact conditions. The analysis matrix is shown in Table 4.

		Impact Conditions									
	Test Ar	ticles		P	Post		Mount		Impact Speed (mph)		
			Weld	Length	Post	Tube Size	Length	Weight	Test	FEA	
Design	Case	lest No.	Size	(in)	Stiffener	(in)	(in)	(1b)			
1	Baseline	15009N	-	51	-			2,372	15.0	15.0	
2	14A-15m	15009G	1/2"	56	А	14x6x25	15	2,372	15.9	15.0	
3	14B-15m	150091	1/2"	56	В	14x6x25	15	2,372	14.1	15.0	
4	12A-15m	15009J	1/2"	54	А	12x6x25	15	2,372	15.1	15.0	
5	12B-15m	15009K	1/2"	54	В	12x6x25	15	2,372	14.9	15.0	
6	14B-21m	15009F	1/2"	56	В	14x6x25	21	2,372	15.2	15.0	
7	12A-21m	15009H	1/2"	54	A	12x6x25	21	2,372	14.5	15.0	

Table 4. Test matrix for validation analysis.

A - Vertical stiffener plates

B - Horizontal stiffener plates

Results

A summary of results is shown in Table 5, which includes a comparison of peak force and post deflection for each case. Comparisons of the force-deflection curves as well as snapshots of the impact event at 0.06 seconds of the impact event are shown in Figures 22 through 28. Because of the oscillatory nature of the data in the FEA results, the values for peak load were estimated based on the average of the force values between peaks. The low frequency oscillations in the FEA results were caused by the choice of material characterization for the wooden head on the front of the pendulum. This component "pulsed" during the analysis while in contact with the post with a frequency corresponding to its elastic stiffness. The wooden head of the pendulum in the physical test crushed slightly during impact but did not show this low frequency behavior as shown in the force-history curve. Although the FEA data contains these low frequency pulses, the overall response of the model can still be adequately discerned from the results.

The force-displacement response of the model matched very well with the physical tests in almost all cases. The one exception was for Design 2 (Case 14A-15m) in which the weld broke during the test resulting in additional displacement of the post. However, the model matched well with the test up to the point when the weld broke, as shown in Figure 23.

The quantitative agreement between the force-time histories of the FEA and physical tests were also computed and are included in Table 5 as well as on the force-time plots in Figures 22 through 28. The RSVVP software was used to compute the Sprague and Geers (S-G) and ANOVA metrics to measure the similarity between the force-time histories of the analyses and tests. Recall that for full-scale crash tests which have considerable spread in test repeatability the Sprague-Geers metrics should be less than 40%, and the ANOVA metrics should have a mean of less than 5 percent and a standard deviation of less than 35 percent. For the impact case used in this study (i.e., rigid pendulum into post-mount), the test set-up was very simple and the repeatability of test results were expected to be relatively high compared to full-scale crash tests. There are no defined acceptance criteria for these types of tests, so the quantitative metric values are reported simply for completeness. However, the results of the quantitative metrics confirm that the general shape (magnitude and mean) of the test and analysis curves are in very good agreement (i.e., low values) for all cases except for Design II; while the phase and the standard deviation are in less agreement (i.e., higher values) for all cases due to the low frequency oscillations of the data caused by the pendulum model. Additional results and comparisons are provided in Appendix K.

			2,372-lb P	endulum	Results											
Test Articles			Speed	(mph)		Peak Force								Peak disp.		
			Test		Test	FEA	Error	Sprague	-Geers	ANG	OVA	Test	FEA	Error		
Design	Case	Test No.	Test	FEA	(kips)	(kips)	(%)	magnitude	phase	mean	std. dev.	(in)	(in)	(%)		
1	Baseline	15009N	15.0	15.0	28.2	33	17.0	1.9	8.2	-1.3	17.9	8.8	8.0	-8.6		
2	14A-15m	15009G	15.9	15.0	30.9	34.1	10.4	24.2	13.4	16.2	28.1	12*	8.6	N.A.		
3	14B-15m	150091	14.1	15.0	30.2	34.6	14.6	-0.1	6.6	-0.9	15.9	9.2	9.1	-1.4		
4	12A-15m	15009J	15.1	15.0	28.3	30.4	7.4	7.4	6.8	4.6	18	9.4	9.3	-1.1		
5	12B-15m	15009K	14.9	15.0	27.8	30	7.9	2.9	7	1.8	18.1	9.7	9.7	0.0		
6	14B-21m	15009F	15.2	15.0	30.6	32	4.6	0.4	7.7	0.3	16.4	8.8	9.3	5.7		
7	12A-21m	15009H	14.5	15.0	28.1	31	10.3	-5	6.7	-2.9	14.6	9.3	9.4	1.1		

Table 5. Summary of FEA vs. Test.

A - Vertical stiffener plates

B - Horizontal stiffener plates

* Weld ruptured just as the pendulum was closing to a stop, allowing additional deflection.



Figure 22. FEA vs. Test for Design 1 (Baseline).


Figure 23. FEA vs. Test for Design 2 (Case 14A-15m).



Figure 24. FEA vs. Test for Design 3 (Case 14B-15m).



Figure 25. FEA vs. Test for Design 4 with (Case 12A-15m).



Figure 26. FEA vs. Test for Design 5 (Case 12B-15m).



Figure 27. FEA vs. Test for Design 6 (Case 14B-21m).



Figure 28. FEA vs. Test for Design 7 (Case 12A-21m).

Conclusions Regarding Model Validity

The finite element model of the various post-mount designs was validated by comparing the analysis results to those of physical impact tests performed at the FOIL in Test Series 15009. The following conclusions have been drawn regarding the validity of the finite element model.

- 1) From a general comparison of the sequential snapshots of the test and simulation, the general response of the FE model appears reasonable with regard to the basic deformations of the post and mount for the given impact conditions.
- 2) The comparison of the force-displacement curves indicated that the finite element models replicated the tests very well regarding magnitude and overall shape of the curves.
- 3) The quantitative measurements to determine similarity between the analyses and physical tests also confirmed that the model was sufficiently valid. It is believed that the primary differences in the curves were primarily due to inadequate material characterization of the pendulum head.
- 4) Thus, the finite element models for the W6x25 post, the mounting tube, the mounting plates and the hardware are all considered valid for use in applications where the post-mount is subjected to similar or lower levels of deformation.

Evaluation of Modified Post-Mount Designs Including Stiffness Effects of Bridge Structure Using FEA

The focus of this chapter is on the development of detailed finite element models of several bridge superstructure designs and conducting finite element simulations of pendulum impacts for selected post-mount design options. The purpose for the analyses was to further investigate the stiffness response of the proposed mount designs presented in the previous chapters and to conduct a more critical evaluation of the resulting loads and stresses imposed on the superstructure. The goals of this task were (1) to determine applicable post-mount design(s) for a given bridge structure and (2) to determine the minimum size requirements for critical bridge components to ensure that deformations of the bridge structure are negligible and/or acceptable under impact loading of the bridge rail.

A series of preliminary analysis cases were performed to identify trends related to sizing of the critical bridge components that either (1) minimize the amount of strain for <u>all</u> bridge components or that (2) force the damage onto a single component that could easily be repaired or replaced, such as the structural Tee component that connects the web of the fascia beam to the interior diaphragm elements. Based on the results of that study, several candidate post-mount-and-bridge designs were selected for further evaluation, which are listed in Table 6. The complete results for the preliminary evaluations study can be found in the technical report prepared for the Ohio DOT by Plaxico *et al.*[*Plaxico15b*] The following sections of this chapter present detailed evaluations of these designs using a more detailed finite element model.

	Fascia Beam		Diaphragm			WT Connector			Mou		Post		
		Thick	ness		Thick	kness		Thick	kness		Thicknoss	Longth	Fust-
Analysis	Size	Flange	Web	Size	Flange	Web	Size	Flange	Web	Size	(in)	(in)	Design
Case		(in)	(in)		(in)	(in)		(in)	(in)		(,	(,	Design
А	W14x30	0.385	0.27	C12x25	0.501	0.387	WT6x36	0.67	0.43	TS12x6x0.25	0.25	15	A (vert. plates)
В	W14x48	0.385	0.27	C10x20	0.436	0.379	WT6x32.5	0.605	0.39	TS12x6x0.25	0.25	15	A (vert. plates)
С	W14x48	0.385	0.27	C10x20	0.436	0.379	WT6x32.5	0.605	0.39	TS12x6x0.25	0.25	15	B (horz. plates)
D	W16x40	0.385	0.27	C12x20.7	0.501	0.282	WT6x32.5	0.605	0.39	TS14x6x0.25	0.25	15	A (vert. plates)
E	W16x40	0.385	0.27	C12x20.7	0.501	0.282	WT6x32.5	0.605	0.39	TS14x6x0.25	0.25	15	B (horz. plates)
F	W16x40	0.385	0.27	C12x25	0.501	0.387	WT6x32.5	0.605	0.39	TS14x6x0.25	0.25	15	B (horz. plates)

Table 6. Study matrix for post-mount designs attached to bridge superstructures.

Finite Element Model of Bridge Structures

Finite element models for each of the bridge-and-post-mount designs listed in Table 6 were developed. The models included a fascia beam and two rows of interior stringer beams modeled as steel wide flange sections spaced at 4 feet on centers. The diaphragm elements between the longitudinal fascia and stringer beams were modeled as steel C-channel sections spaced at 6.25 feet on centers and positioned at each post-mount location. The diaphragms were connected to the web of the fascia beam using a structural Tee section and to the web of the interior stringers using a structural Angle section. The bolted connections of the Tees and the angles were modeled with 7/8" diameter bolts with properties of A325-1 steel. The bolts and washers were model explicitly using the same methodology described in Chapter 8 for the post mount connections. The deck was modeled as a 3" x 9" 5-gauge corrugated steel tack welded to the top of the stringers. The tack welds were modeled using the spot-weld option in LS-DYNA (no failure conditions were included for the spot-welds). The asphalt overlay was not included in the analysis.

The bridge was modeled 31.25 feet long and 12.5 feet wide (e.g., approximately half the bridge width). The 31.25 feet length is typical for rural roadway applications. Translational constraints were applied at the lower flange at each end of the stringer beams and at the bolt holes in the angle brackets located at the interior boundary of the bridge model.

The post-mount system was connected to the bridge structure using six 7/8" diameter bolts (i.e., three on each side of the mount) with the bolts passing through 1" diameter holes in the tubular mounting plate, the web of the fascia beam and the flange of the WT-connector. The bolts were modeled using beam elements in LS-DYNA with material properties corresponding to ASTM A325-1 and were pre-tightened to approximately 12 kips. The finite element model for analysis Case C is shown in Figure 29, and a close up view of the model is shown in Figure 30. The element size for the bridge structure was $\frac{1}{2}$ " x $\frac{1}{2}$ " in regions near component connections and 1"x1" elsewhere. The mesh for the corrugated deck was modeled as 2"x2" (typical).



Figure 29. Typical model for bridge superstructure and post-mount used in the analyses.



Figure 30. Close-up view of finite element mesh.

Loading Conditions

The loading on the post was applied at 35.75 inches above the centerline of the top bolt of the post-mount corresponding to 25.75 inches above the 8-inch deck, as illustrated in Figure 31. The load was applied using the *Rigidwall_Geometric_Cylinder_Motion option in LS-DYNA with the sinusoidal displacement-time history shown in Figure 31. The 8-inch diameter rigid cylinder struck the post at 21 mph and reached a maximum deflection of 12 inches before unloading. The 12-inch deflection is much greater than the measured deflection of the system under TL4 impact conditions [*Buth93*]. As discussed in Chapter 3, it is possible that the bridge rail will at some point experience impact severities greater than TL4 and it is therefore important to ensure that the loads transferred into the bridge structure during such loading conditions do not result in excessive damage to the bridge superstructure. The same loading condition was also applied to the baseline post-mount model at a height of 28.75 inches above the top mounting bolt (i.e., corresponding to 25.75 inches above the deck surface).



Figure 31. Loading conditions for the modified post-mount and bridge model.

Solution Verification

The first step in the Report W179 validation process is to perform global checks of the analysis to verify that the numerical solution is stable and is producing physical results (e.g., results conform to the basic laws of conservation).[*Ray11*] Table 7 shows a summary of the global verification assessment based on criteria recommended in Report W179. Figure 32 shows a plot of the global energy-time histories from the analysis. This analysis was <u>not</u> modeled as a closed system; that is, the massless rigid cylinder was given a displacement-time history, thus resulting in accrued energy as it pushes the post. The accrued energy, labeled as "external work" in LS-DYNA, should equal the total energy in the analysis, which is the sum of the internal, kinetic, and friction energies, plus the various numerical energies from the analysis (e.g., hourglass).

Verification Evaluation Criteria	Change (%)	Pass?
<i>Total energy</i> of the analysis solution must not vary more than 10 percent from the beginning of the run to the end of the run [from the external work].	0%	Y
<i>Hourglass Energy</i> of the analysis solution at the end of the run is less than <i>five percent</i> of the total <i>initial energy</i> at the <i>beginning</i> of the run.	0%	Y
<i>Hourglass Energy</i> of the analysis solution at the end of the run is less than <i>ten percent</i> of the total <i>internal energy</i> at the <i>end</i> of the run.	0%	Y
The part/material with the highest amount of <i>hourglass energy</i> at the end of the run is less than ten percent of the total internal energy of the part/material at the end of the run.	0%	Y
<i>Mass added</i> to the total model is less than five percent of the total model mass at the beginning of the run.	0%	Y
The part/material with the most <i>mass added</i> had less than 10 percent of its initial mass.	0%	Y
The moving parts/materials in the model have less than five percent of <i>mass added</i> to the initial moving mass of the model.	0%	Y
Any <i>shooting nodes</i> in the solution?	N	Y
Any solid elements with <i>negative volumes</i> ?	N	Y

Table 7. Analysis Solution Verification Table.



As shown in Table 7, all the solution verification parameters were satisfied so it can be reasonably assumed that the solution represents a physically plausible loading event that obeys basic conservation laws. This is confirmed in the Figure 32 which shows the total energy (blue curve) is coincident with the external work (orange curve). The solution meets all the recommended global energy balance criteria and appears to be free of any major numerical problems. This does not indicate that the simulation is necessarily valid, only that the results adhere to the basic laws of physics and that the solution is numerically stable.

Results

Force-Deflection

The force-deflection responses for each of the analysis cases in **Table 6** are compared to the baseline case in Figure 33. The vertical dashed line on the plot indicates the maximum deflection of the rail in full-scale crash test 7069-37, which was estimated from the test videos to be 4 inches. The horizontal dashed line on the plot indicates the minimum TL4 loading on the post calculated earlier using the LRFD equations to be 24 kips. The force-deflection response for all the fascia mount designs was equivalent to or stiffer than the baseline case up to 4 inches of deflection of the post. Beyond 4 inches of deflection, Cases D, E and F (i.e., the W16x40 fascia beam designs) resulted in equivalent or stiffer response than the baseline, while Cases A, B, and C (i.e., W14x30 and W14x48 fascia beam designs) resulted in lower stiffness. However, the peak forces in all cases exceeded the minimum TL4 force requirement of 24 kips. Based on these assessments the modified-post-mount-and-bridge designs are expected to provide equal or better stiffness for the bridge rail system in NCHRP Report 350 TL4 crashes, and thus should result in equivalent or better crash performance for those cases.



Figure 33. Force-Deflection for each of the analysis cases compared to the baseline case.



Figure 34. Images showing deformations of post-mount and bridge at 4 inches post deflection (e.g., TL4 loading).



Figure 35. Images showing deformations of post-mount and bridge at 12 inches post deflection.

Plastic Deformation

The plastic strains in the fascia beam, diaphragm, and the Tee connector were evaluated to assess the level of damage in the bridge structure. Figure 36 shows sequential views of deformation for the post-mount-and-bridge structure for Analysis Case A with contours of effective plastic strain. Figures 37 through 39 show contours of effective plastic strain, 1st principal strain and maximum shear strain, respectively, for the critical bridge components at 4 inches of post deflection. Figures 40 through 42 show contours of effective plastic strain, 1st principal strain and maximum shear strain, respectively, at 12 inches of post deflection. The range for the contour scale is cut-off at 0.01 in/in to assist visual identification of critical strain regions. The maximum strain values for the individual components are also labeled on each plot. Images of the strain contours for all the analysis cases are listed in Table 8 and Table 9 for the effective plastic strains and 1st principal strains, respectively. These strains are also plotted in Figures 43 through 46.



Figure 36. Analysis Case A showing deformation and contours of 1st Principle Strain at 4" post deflection, 12" post deflection and after unloading.



Figure 37. Contours of effective plastic strain for Analysis Case A at TL4 loading conditions.



Figure 38. Contours of 1st principal strain for Analysis Case A at TL4 loading conditions.



Figure 39. Contours of maximum shear strain for Analysis Case A at TL4 loading conditions.



Figure 40. Contours of effective plastic strain for Analysis Case A at peak loading conditions.



Figure 41. Contours of 1st principal strain for Analysis Case A at peak loading conditions.



Figure 42. Contours of maximum shear strain for Analysis Case A at peak loading conditions.

	Case A (W14x30)		Case B (W14x48)		Case C (W14x48)		Case D (W16x40)		Case E (W16x40)		Case F (W16x40)	
Component	4"	12"	4"	12"	4"	12"	4"	12"	4"	12"	4"	12"
Fascia Beam	0.0067	0.0428	0.0078	0.0422	0.0079	0.039	0.0107	0.0546	0.0107	0.0502	0.0109	0.0536
Diaphragm	0.0358	0.0822	0.044	0.08698	0.0232	0.0483	0.0618	0.1053	0.0592	0.1	0.0388	0.0631
WT Flange	0.0162	0.0417	0.0115	0.03945	0.013	0.0408	0.01711	0.0492	0.0173	0.0465	0.0173	0.0507
WT Web	0.0345	0.0779	0.04511	0.085	0.0246	0.0703	0.0316	0.0476	0.029	0.0456	0.0388	0.0638

Table 8. Values of effective plastic strain in critical bridge components at 4.6" (TL4) and 12" (peak) post deflection.

Table 9. Values of 1st principal strain in critical bridge components at 4.6" (TL4) and 12" (peak) post deflection.

	Case A (W14x30)		Case B (W14x48)		Case C (W14x48)		Case D (W16x40)		Case E (W16x40)		Case F (W16x40)	
Component	4"	12"	4"	12"	4"	12"	4"	12"	4"	12"	4"	12"
Fascia Beam	0.0008	0.0069	0.0001	0.0049	0.0006	0.0048	0.0012	0.0081	0.0012	0.0071	0.0009	0.0085
Diaphragm	0.0160	0.0344	0.0173	0.0350	0.0124	0.0201	0.0254	0.0467	0.0246	0.0441	0.0151	0.0246
WT Flange	0.0004	0.0018	0.0005	0.0021	0.0004	0.0022	0.0004	0.0030	0.0004	0.0026	0.0004	0.0032
WT Web	0.0100	0.0236	0.0151	0.0284	0.0073	0.0212	0.0101	0.0166	0.0098	0.0158	0.0126	0.0215





Figure 43. Effective plastic strains in critical bridge components at TL4 loading conditions.

Figure 44. Effective plastic strains in critical bridge components at peak loading conditions.







Figure 46. 1st principal strains in critical bridge components at peak loading conditions.

The following information was deduced from the analysis results:

- The strains in the fascia beam were relatively low for all cases and should not require repair, even for the peak loading case of 12 inches post deflection.
- The most significant plastic strains in the bridge structure occurred at the edges of the bolt holes in the web of the diaphragm and Tee connector caused by the bearing load from the bolts. The highest value of effective plastic strain for the peak loading cases was 0.1 on these components. For A572 Grade 50 steel the strain at the onset of necking in tensile coupon tests is approximately 0.2 and the failure strain is approximately 0.4 (refer to Figure 47).



Figure 47. Stress –strain curve for A572 Grade 50 steel.

• The strains in the diaphragm and the Tee connector were primarily compressive and were localized to the region around the bolt holes. However, the stretching of the bolt holes may require repair or replacement of the components in severe loading conditions. This damage could be reduced by using larger bolts (e.g., 1 inch diameter) and/or adding an additional bolt near the top of the connection to help carry the load.

Conclusions Regarding Post Loading and Size Requirements for Superstructure

The purpose of this chapter was to further investigate the stiffness response of the proposed mount designs and to conduct a more critical evaluation of the resulting loads and stresses imposed on the superstructure. Table 10 shows the post-and-mount design specifications, as well as the minimum size requirements for the bridge diaphragm and Tee connector elements, for various fascia beam sizes as determined from this study. Note that the bridge components listed in Table 10 are minimums, thus larger structural sections may be used. However, the W14x30 is the smallest structural section that can be used for the fascia beam in order to maintain adequate stiffness of the bridge rail without incurring excessive damage to the bridge superstructure.

Fascia Beam			Diaphragm			WT Connector			Mounting Tube				
	Min. Thickness			Min. Thickness			Min. Th	ickness		Thicknoss	Longth	Post Stiffonor Dosign	
Size	Flange	Web	Size	Flange	Web	Size	Flange	Web	Size	(:)	Length	Post-Stillener Design	
	(in)	(in)		(in)	(in)		(in)	(in)		(IN)	(IN)		
\M/1/1×30 - \M/1/1×/3	0 385	0.27	C12v25	x25 0.501 0.387 WT6x36 0.67 0.43 TS12x6x0.25 0.25	15	А	В						
VV14X50 - VV14X45	0.565	0.27	C12725		0.587	******	0.07	0.45	1312×0×0.23	0.25	15	(vert. plates)	(horz. plates)
W14x48 and larger	0.505	0.24	C10-20	0.420	0 270	млс	0.005	0.20	TC124640.25	0.25	15	А	В
W14 sections	0.595	0.34	C10x20	0.436	0.379	W16X32.5	0.005	0.39	1S12x6x0.25	0.25	15	(vert. plates)	(horz. plates)
W16x40 and larger		0.205	C12v20 7	0 501	0 202		0.605	0.20	TC144640 2E	0.25	15	А	В
section sizes	0.505	0.305	C12X20.7	0.501	0.282	VV10X32.5	0.005	0.39	1314x0X0.25	0.25	12	(vert. plates)	(horz. plates)

Table 10. Minimum size for bridge components corresponding to the specified post-mount design.

Design and Evaluation of Release Mechanism for Post Mount under Critical Loading

Mounting-Bolt Design

The post mounting bolts are to be sized such that they provide adequate strength for TL4 impact cases but will fail prior to excessive loads being transferred onto the bridge superstructure. The peak tensile forces of the bolts fastening the tube-mounting-plate to the fascia beam and the bolts fastening the post mounting plate to the tube mounting plate are shown in Table 11. The forces correspond to a peak deflection of 12 inches of the post with loading applied at 25.75 inches above the deck. The locations for the bolts are identified in Figure 48. The bolts used for the fascia beam mount were 7/8-inch diameter A325-1, and those used for the post mounting plate were 1-inch diameter A325-1.

	Fa	scia Mou	nting-Plate	Bolts	Post Mounting Plate Bolts					
	Fo	rce	Stress	s (ksi)	Force	Stress (ksi)				
Analysis	(ki	ps)	7/8"	7/8" 7/8"		7/8"	1"	1-1/8"		
Case	Upper	Mid	Upper	Mid	Upper	Upper	Upper	Upper		
А	53.1	35.3	115	76	88.7	192	146	116		
В	52	30	113	65	91.3	198	151	120		
С	52.6	29.5	114	64	89.5	194	148	117		
D	54.6	35.6	118	77	95.7	207	158	125		
E	54.2	34.9	117	76	90.1	195	149	118		
F	53.5	34.6	116	75	85.8	186	142	112		
			A325-1 Bolt	s		A325-1 Bolts				

Table 11. Forces and resulting stresses for mounting plate bolts.



Figure 48. Bolt identification for Table 11.

The yield strength and ultimate strength for A325-1 bolts are 92 ksi and 120 ksi, respectively. It is assumed that the primary failure mode of the bolts will be in tension and will occur in the bolt threads. The stress area for the bolts was calculated using the following equation:

stress area =
$$0.785 \left(D - \frac{0.9743}{n} \right)^2$$

where $D \equiv bolt \ diamter$
 $n \equiv number \ of \ threads \ per \ inch$

Where *n* equals 9 threads per inch for 7/8-inch diameter bolts, 8 threads per inch for 1-inch diameter bolts, 7 threads per inch for 1-1/8-inch diameter bolts, and 12 threads per inch for 1-inch diameter UNF bolts.

It is important that the bolts which fasten the tube-mount to the fascia beam do not fail. Failure of those bolts would result in disconnecting the diaphragm from the fascia beam and possibly compromise the bridge superstructure. It is therefore necessary that the release of the post occur due to separation of the front mounting plates at the post-side of the mount. The analyses indicated that for 12-inches deflection of the post (i.e., measured at the impact point) the forces in the upper fascia beam mounting bolts were below failure specifications for the A325-1 bolts with peak stresses ranging from 113 to 118 ksi.

The axial forces in the mounting bolts at the front of the post-mount ranged from 86-96 kips. For the 1-inch diameter bolt this corresponded to a peak stress of 142 - 158 ksi, which exceeds the failure stress for the A325-1 bolt; while for the 1-1/8-inch diameter bolt, the resulting peak stress ranged from 112 - 125 ksi, which was relatively low for the A325-1 bolt. It should be noted here that these stress values correspond only to the axial force in the bolt and do not account for the bending stresses that also occur – particularly for the bolts at the front mounting plates. Additionally, these stress values correspond to the force in the bolt at 12-inches deflection of the post.

Recall that the deflection of the post to achieve TL4 performance was determined from full-scale testing and LRFD calculations to be between 4 and 6 inches, and the maximum post deflection to avoid damage to the bridge superstructure was defined as 12 inches. Based on those factors, the failure condition for the releasing the post from the mount was therefore set to 8 - 11 inches of post deflection.

Figure 49 shows a typical axial force response for the top mounting bolt of the post mounting plate versus the lateral deflection of the post at the impact point for Analysis Case E. The stress in the bolt at 8-inches deflection is approximately 9 percent less than the stress at 12-inches deflection. It was therefore decided that the 1-inch diameter A325-1 bolt would be the best candidate for providing sufficient strength for TL4 loading, but also meeting the desired failure conditions. In order to confirm that this bolt meets the desired failure conditions for the post-mount, physical testing (similar to the previous test program) was performed on the two post mount designs (i.e., HSS 14x6x3/8 and HSS 12x6x3/8); those results are presented in the following section.



Figure 49. Axial force in post-plate mounting bolt versus post deflection for analysis Case E.

Weld Design

The design weld size for the post-to-plate joint was determined using simple strength of materials calculations based on a design load of 145 kips (determined from finite element analysis), a weld strength of 60 ksi, and weld length of 5.5 inches. Under these loading conditions the weld typically fails in shear, through the throat of the weld. The designed throat-thickness of the weld was calculated to be 7/16-inch, as shown in Figure 50, which was rounded up to ½-inch throat-thickness for the final design. This corresponds to a leg-length of 5/8 inch for the fillet weld. Also, shown in Figure 50 are the weld size calculations using LRFD methods. These calculations include safety factors that result in oversizing the weld to prevent weld failure; however, in this application the failure of the weld is an inherent part of the design.



Figure 50. Weld size calculations based on strength of materials approach versus LRFD design approach.

Dynamic Pendulum Testing of Failure Mechanism Design

Dynamic pendulum impact tests were again performed on the post-mount design to verify that the post-mounting bolts and the weld-joint between the post flange and post-mounting-plate provide the required strength and failure characteristics for the mount. The tests were conducted at the Federal Outdoor Impact Laboratory (FOIL) at the Turner-Fairbank Highway Research Center located in McLean, Virginia. The physical testing performed in this task was limited to pendulum impact on two post-and-mount designs (i.e., HSS 14x6x3/8 and HSS 12x6x3/8) with the same basic test setup used in the previous test program (refer to Section titled *Strength Assessment of Proposed Design Options via Pendulum Tests*). The impact conditions were defined to achieve a target displacement for the post of approximately 10-12 inches.

Test Articles

Two post-mount designs were included in the tests, as shown in Figure 6. Design Type I included an HSS 14"x6"x ¹/4" and design Type II included an HSS 12"x6"x ¹/4". Both designs included the Type B post-stiffeners composed of four steel plates ¹/4" thick, two on each side of the post, fitted horizontally between the post's flanges. The two top stiffener plates were positioned at 5 inches above the top of the tube mount and the two lower stiffener plates were positioned at ¹/4 inch above the top of the tube mount. In both designs, a 1" thick plate was welded onto the front flange of the post to enable connection to the mount structure. The length of the HSS tube mounts was 15 inches. The materials (including hardware), fabrication and shipping of the test articles were provided by U.S. Bridge. The detailed shop drawings for the test articles are provided in Appendix L.



Figure 51. Drawings for the two post-mount designs tested.

Test Matrix

The test setup was essentially the same as that of the previous test program in Test Series 15009 (refer to section titled *Strength Assessment of Proposed Design Options via Pendulum Tests*). The test matrix for this study, shown in Table 2, included 10 tests: five tests for design Type 1 and five tests for design Type II. The complete test program included additional test cases involving various weld sizes and bolt types, but only those corresponding to the final design options are included herein. A summary of the complete test program results is included in Appendix M and Appendix N.

The test matrix included four tests on each mount type using 1-inch diameter A325-1 bolts at the top-bolt position of the plate-to-plate connection between the post and mount (according to the proposed design); and one test on each mount type using 7/8-inch diameter A325-1 bolt at the top-bolt position. The purpose for performing the test cases involving the 7/8-inch diameter bolts was based on a concern that the 7/8" bolt, which is the standard bolt size used on most steel bridges, may inadvertently be installed on the post-mount during construction

in the field. It was therefore decided by the TAC that the consequences of such a mistake be clearly understood so that the correct plan of action could be implemented to avoid it (e.g., circumvent by preassembling the post-and-mount in the shop prior to shipping to the field site).

The 1-inch diameter top mounting bolts were tightened according to the standard specifications for bolt tightening on bridge structures using the turn-of-nut pretension method, which states that the rotation of the nut from snug-tight condition should be *not more than 1/3 turn*. It was difficult, however, to identify the correct snug-tight starting point for the nut at the test site, so a torque wrench was used to tighten the top mounting bolts to 300 ft-lb which resulted in approximately 1/3 turn for all cases.

Two different pendulum masses were used in the study, as illustrated in Figure 52. The smaller pendulum weighed 2,372-lb and the heavier pendulum weighed 4,500-lb. Impact conditions were defined to achieve approximately 28 kip-ft of energy; which resulted in target impact speeds of 18.8 mph and 13.6 mph for the small pendulum and large pendulum, respectively. These impact conditions were determined through FEA to be sufficient for rupturing the top mounting bolts.

		Top Mounting Bolts			Imp	act Conditi	ons	Result				
Mount Design	Test #	Diameter	Туре	Post-Plate Weld Size	Mass (kg)	Speed (mph)	Energy (kip-ft)	Max Force* (kips)	Disp at Failure (in)	Energy Absorbed (kip-ft)	Failure Mode	
	16022A	1"	A325-1	5/8"	2,372	19.0	28.6	33.3	10.1	23.9	Bolt Rupture	
Type 1	16022B	1"	A325-1	5/8"	2,372	18.9	28.3	33.4	9.7	23.2	Bolt Rupture	
(14")	16022G	1"	A325-1	5/8"	4,500	13.1	25.8	39.3	N.A.	24.8	-	
	16022H	1"	A325-1	5/8"	4,500	13.7	28.2	39.9	8.1	22.4	Bolt Rupture	
	16016G	7/8"	A325-1	5/8"	2,372	16.0	20.3	34.0	6.2	13.3	Bolt Rupture	
	16022C	1"	A325-1	5/8"	2,372	18.7	27.7	30.2	8.4	18.8	Bolt Rupture	
Turno 2	16022D	1"	A325-1	5/8"	2,372	19.0	28.6	30.4	9.0	19.9	Bolt Rupture	
(12")	16022E	1"	A325-1	5/8"	4,500	12.9	25.0	31.7	9.3	24.3	Weld	
	16022F	1"	A325-1	5/8"	4,500	13.5	27.4	31.5	9.5	25.0	Weld	
	16016B	7/8"	A325-1	5/8"	2,372	16.0	20.3	28.4	5.8	11.3	Bolt Rupture	

Table 12.	Test matrix for	evaluating post-mount	connection strength	and failure characteristics.
		91	0	

* Maximum force occuring after inertial peak



Figure 52. Pendulum impactors used in the impact tests.

Test Results

A summary of the test results is shown in Table 12 including peak force, post deflection at failure, energy absorbed, and the characteristic mode of failure. Figure 53 shows the force-

deflection results for post-mount Type I (i.e., the 14" x 6" x 0.25" tubular mount); Figure 54 shows the force-deflection results for post-mount Type II (i.e., the 12" x 6" x 0.25" tubular mount); and Figure 55 shows the force-deflection results for post mount Types I and II with a 7/8-inch diameter bolt in the top-bolt position. As in the previous test program results, the initial force spike (inertial) was not considered when determining the maximum force values.



Target Displacement Range

Figure 53. Test results for post-mount Type I (14" x 6" x 0.25").



Figure 54. Test results for post-mount Type II (12" x 6" x 0.25")



Figure 55. Test results for post-mount Types I and II with 7/8" diameter top-mounting bolts.

As shown in Figures 53 and 54, the response for post-mount designs Type I and II both met the target force and deflection criteria (i.e., force > 27 kips and post deflection of 8 - 11 inches). The primary mode of failure was tensile rupture of the 1-inch diameter A325-1 bolts at the top-bolt position, as shown in Figure 57(a); however, rupture of the welded-joint between the post-flange and the 1-inch thick post-mount plate occurred in two of the test cases (i.e., 16022E and 16022F), as illustrated in Figure 57(b). This indicated that the 5/8-inch weld size and the 1-inch diameter A325-1 bolts provide similar failure strength for the post-to-mount connection; thereby, providing a second level of security for the bridge.

Finite element analyses were performed that simulated the exact test conditions for Tests 16022A (Type I mount) and 16022C (Type II mount). The FEA model included a failure strain condition for the bolts set to 0.07 in/in. The force-time history and the force versus deflection response for the FEA is compared to the test results in Figure 56, which shows that the model adequately replicates the physical response.



Figure 56. Force vs. time and force vs. deflection results, comparing FEA with Tests.

For the tests in which the 7/8-inch diameter bolts were used in the top mounting bolt location, the connection strength did not meet the desired failure conditions but did meet minimum conditions for NCHRP report 350 TL4, as shown in Figure 55. The force magnitude exceeded 27 kips and the post deflection exceeded 6 inches. Thus, if the 7/8-inch diameter bolts are inadvertently installed at one or more post mount locations, the strength of the mount will be degraded but should be adequate (e.g., not fail) for most impact conditions.





(a) Bolt rupture





(b) Weld rupture

Figure 57. Typical failure modes for the post-mount connections.

Report 350 Test 4-12 Simulation on Modified Illinois Bridge Rail System

Test 4-12 was simulated for the modified Illinois Two-Tube bridge rail with impact conditions similar to the original full-scale test to verify that the modifications do not adversely affect crash performance. The analysis was also used to ensure that all loading conditions for the post and mount have been accounted for in its design. A detailed finite element model of the bridge rail was developed using the modified mount with Post Stiffener Type A. An asphalt deck was also included in this model with an overall depth of 8 inches. The bottom nodes of the asphalt mesh were merged with those of the corrugated steel deck to create the bond between the two components.

The impact conditions for the analysis corresponded to those of full-scale crash test 7069-37 in which an 18,000-lb SUT impacted the bridge rail 2.3 feet downstream of a bridge rail post at 51.4 mph at 15 degrees. The total length of the test article was 84 feet. For the analysis, the bridge model developed in section *Bridge Structure Loading and Damage Assessment* was used which included 31.2 feet of the bridge structure. The ends of the bridge stringer beams were modeled with fixed constraints in the x, y, and z direction. The bridge railing was extended both upstream and downstream of the bridge to simulate continuation of the rail. The bottom of the posts in the transition sections were also modeled with fixed constraints. Figure 58 shows a side view of the overall model; while detailed views are shown in Figure 59.



Figure 58. The finite element model used for the Test 4-12 simulation.



Figure 59. Detailed views of bridge and bridge rail model.

Vehicle Model

The vehicle model used in analysis was the 8000S single unit truck model developed at the National Crash Analysis Center (NCAC) in Ashburn, VA which has been further modified by various researchers over the years to improve its fidelity in analysis of impact conditions corresponding to *NCHRP Report 350* Test 4-12. [*Miele05*; *Mohan07*; *Plaxico13*] The model of the ballast was modified in order to calibrate the mass inertial properties of the vehicle model to the properties of the test vehicle. The ballast was modeled as a rigid block with dimensions of 48 inches wide x 52 inches long x 30.5 inches tall. The exact type and overall geometry of the ballast used in the full-scale test was unknown, but the crash test videos showed sand pouring from the front of the box during the test.

A comparison of the dimensional and inertial properties of the test vehicle and FE model is shown in Figure 60. The most notable discrepancy in the dimensional properties (aside from the type and size of ballast used) was the c.g. height. However, the value provided in the test report corresponded to the location of the accelerometer and not the actual center of mass for the vehicle.[*Buth93*]



Figure 60. Comparison of vehicle properties for FE model and test vehicle.

Simulated Impact Summary

The finite element analysis was conducted with a time-step of 1.0 microsecond for a time period of 0.8 seconds. The acceleration-time history results and the angular displacement results for the analysis of the modified system and from the full-scale test on the original system are shown in Figures 61 through 66. Sequential views of the analysis and the test are shown in Figures 67, 68 and 69 from a downstream viewpoint, side viewpoint and overhead viewpoint, respectively. Based on visual inspection the model appeared to simulate the basic kinematic behavior of the truck and adequately captured the basic phenomenological events that occurred in the test.

At 0.02 seconds after impact, the front left tire of the vehicle model contacted the bridge rail, which was consistent with the test. At 0.06 seconds the vehicle model began to redirect and the front axle began to slide along the leaf springs. From the test video and the acceleration plots, it appeared that the u-bolts that fastened the front axle to the leaf springs of the test vehicle ruptured very soon after the front tire impacted the rail (note the sudden acceleration drop at 0.06

seconds in the x-acceleration in Figure 61). At 0.11 seconds the front axle of the vehicle model contacted the rear shackle of the leaf spring, stopping further sliding of the axle. At 0.14 seconds the u-bolts holding the front axle to the leaf spring failed. At 0.20 seconds the front impact-side wheel of the vehicle model jammed into the wheel-well causing a deceleration spike in the longitudinal direction. The rear tandems of the vehicle model contacted the rail at 0.285 seconds and the vehicle was traveling at 42.9 mph. In the full-scale test the rear tandems contacted the rail at 0.283 seconds and the vehicle was traveling at 47.8 mph. The bridge rail reached a maximum dynamic deflection of 4.3 inches at 0.33 seconds. The peak dynamic deflection was not measured in the full-scale test. At 0.355 seconds of the analysis the front corner of the cargobox set down on top of the rail. In the full-scale test the box set down on the rail at 0.331 seconds. At 0.46 seconds the rear tandems started to rebound away from the rail. The analysis was terminated at 0.8 seconds, at which time:

- The roll angle of the cargo-box was 20 degrees and decreasing,
- The pitch angle of the truck cabin was 3.9 degrees and starting to decrease,
- The yaw angle of the truck cabin was 1 degree (toward the barrier) and constant,
- The forward velocity of the vehicle was 58.2 mph.



Figure 61. X-acceleration time-histories from the c.g. of the vehicle for the modified system (FEA) and the original system (Test) under Test 4-12 impact conditions.



Figure 62. Y-acceleration time-histories from the c.g. of the vehicle for the modified system (FEA) and the original system (Test) under Test 4-12 impact conditions.



Figure 63. Z-acceleration time-histories from the c.g. of the vehicle for the modified system (FEA) and the original system (Test) under Test 4-12 impact conditions.



Figure 64. Roll angle time-histories from the c.g. of the vehicle for the modified system (FEA) and the original system (Test) under Test 4-12 impact conditions.



Figure 65. Pitch angle time-histories from the c.g. of the vehicle for the modified system (FEA) and the original system (Test) under Test 4-12 impact conditions.



Figure 66. Yaw angle time-histories from the c.g. of the vehicle for the modified system (FEA) and the original system (Test) under Test 4-12 impact conditions.



Figure 67. Sequential views for FEA and Test 7069-37 from downstream viewpoint.



Figure 67. [Continued] Sequential views for FEA and Test 7069-37 from downstream viewpoint.



0.0 seconds







Figure 68. Sequential views for FEA and Test 7069-37 from side viewpoint.



0.2 seconds





Figure 68. [Continued] Sequential views for FEA and Test 7069-37 from side viewpoint.







Figure 68. [Continued] Sequential views for FEA and Test 7069-37 from side viewpoint.



0.6 seconds



Figure 68. [Continued] Sequential views for FEA and Test 7069-37 from side viewpoint.

0.00 seconds





0.10 seconds

















Figure 69. Sequential views for FEA and Test 7069-37 from overhead viewpoint.


Figure 69. [Continued] Sequential views for FEA and Test 7069-37 from side viewpoint.

Comparing the results from accelerations and the sequential views seem to indicate that the test accelerometer for the analysis may not have been positioned and/or mounted to the truck in the same way that it was in the test. For example, from the angular displacement plots shown in Figure 64, the peak response for the roll angle was considerably higher for the analysis compared to the test (e.g., 26 vs 15 degrees). The sequential views shown in Figure 67, on the other hand, show the opposite – that is, that the roll of the cargo-box for the test vehicle was considerably higher than the analysis. Likewise, the y-accelerations start to diverge at approximately 0.1 seconds, which is when the cargo-box for the test vehicle began its lateral shift, ultimately sliding off the truck's frame-rails. This phenomenon was not possible in the FE model, since there was an additional brace placed near the front of the cargo-box in the model to prevent lateral shifting of the box at that location. The addition of this lateral constraint aids in keeping the truck-bed from sliding off the main frame-rails during impacts and can have a significant effect on the dynamics of the vehicle, as illustrated in Figure 70 for tests on a single-slope concrete barrier with (a) restrained and (b) unrestrained cargo boxes. An example of a typical lateral constraint bracket on a SUT vehicle is shown in Figure 71.

The roll position of the accelerometer mounted to the cargo-box will directly affect the magnitudes of the accelerations in the local y- and z-directions because they measure accelerations in a local coordinate frame. The method of connecting the bed to the truck is not peculiar to vehicle type. For example, the test vehicles shown in Figure 70 are both GMC but have different bed-connection strategies, which lead to different kinematic responses from the vehicle.



Figure 70. Example of tests performed with a) front of cargo-box restrained (TTI Test 7147-17) and b) front of cargo-box unrestrained (TTI Test 7147-16).[*Mak98; Buth97*]

The pitch of the vehicle was more positive in the test (due to the front tire of the test vehicle starting to climb the rail upon impact); however, the overall magnitude of the pitch was less than four degrees for both the test and the analysis which was considered relatively insignificant.

From this general comparison of the test and analysis, it was concluded that the accelerometer in the full-scale test may have been positioned underneath the cargo-box (rather than inside). If so, it would explain why there were notable differences in the time-history data, while the sequential views showed relatively good correlation. In order to verify this, the accelerometer model would need to be revised to match the exact positioning and mounting

design used in the full-scale test. The model of the ballast should also be revised to more closely match the ballast used in the test (e.g., sand bags instead of concrete). Unfortunately, this information was not available in the test report and it could not be determined from the test videos.



Figure 71. C-channel welded to bed-frame and bolted to main frame-rail for lateral support.

Damage to Bridge Rail

The damage to the baseline bridge rail in the full-scale crash test was limited to gouging of the rail by the lug-nuts of the truck wheels, permanent deformation of the angles at Posts 3 through 5, several loose bolts due to the deformation of the rail, and fracture of a bolt-head on one of the bolts on the top rail, as shown in Figure 72. The dynamic deflection of the rail was not reported, but the maximum permanent deflection was reported to be 2.5 inches.

The damage to the modified system in the FE analysis was similarly minor. The dynamic deflection was 4.3 inches and the maximum permanent deflection was approximately 1 inch. The tops of two posts were deformed and the top bolts were bent, as shown in Figure 73, which occurred when the bottom of the cargo-box impacted the top of the railing and snagged the posts.



Figure 72. Damage to baseline bridge rail in full-scale test 7069-37.



Figure 73. Deformations at the top of two posts resulting from the cargo-box snagging on the posts.

Damage to the Bridge Structure

Figure 74 shows sequential views of the deformations of the bridge components at the location of maximum loading (i.e., at Post 2) from an overhead viewpoint. The flanges of the fascia beam and the diaphragm were made transparent to facilitate viewing. The maximum dynamic separation between the mounting plate and the fascia beam was 0.4 inches at 0.33 seconds of the impact event. The maximum permanent separation was 0.12 inches ($\approx 1/8$ inch). Figure 75 shows contours of effective plastic strain in the bridge components at the location and time of maximum loading. The hardware was removed from view in order to show the maximum strains around the bolt holes in the tee connector, the diaphragm and the fascia beam. The Tee connector was removed from view in the third image of Figure 75 to show the strains in the fascia beam web. The maximum plastic strain for the diaphragm and the web of the tee connector was 0.055 and occurred at the edge of the bolt holes. The maximum plastic strain for the flange of the tee was 0.018, and the maximum strain for the fascia beam was 0.01. These values are considered negligible and would thus not require repair or replacement after impacts with similar or less severity. The forces in the bolted connections reached peak values of 33 kips, which corresponds to a maximum stress of 71 ksi for a 7/8-inch diameter UNC bolt. Recall that the yield stress for A325-1 steel is 92 ksi.



Figure 74. Overhead view showing sequential views of maximum dynamic deformation and final permanent deformation of bridge components at location of maximum loading.



Figure 75. Contours of effective plastic strain for bridge components (with bolts removed from view) at location of maximum loading.

Occupant Risk Metrics

Acceleration-time histories and angular rate-time histories were collected at the center of gravity of the truck using the *Element-Seatbelt-Accelerometer option in LS-DYNA. [LSDYNA13] The accelerometers were connected to the vehicle model using *Nodal-Rigid-Body-Constraints (NRB). The time-history data was collected from each accelerometer in a local reference coordinate system which rotates with the accelerometer in the same way that test data is collected from physical accelerometers. The data was collected at a frequency of 30 kHz which was determined to be sufficient to avoid aliasing of the data in the numerical model.

The occupant risk assessment measures were computed using the x-, y- and zacceleration time-histories and angular-rate time histories collected at the center of gravity of the vehicle. The Test Risk Assessment Program (TRAP) calculates standardized occupant risk factors from vehicle crash data in accordance with the National Cooperative Highway Research Program (NCHRP) guidelines and the European Committee for Standardization (CEN).[TTI98]

Table 13 shows the occupant risk measures from the full-scale test on the baseline system and the quantities computed from the FEA analysis for the modified system. The acceleration data from the analysis was filtered using the SAE Class 180 filter prior to input in TRAP. The table shows the two occupant risk factors recommended by NCHRP Report 350, which include: 1) the lateral and longitudinal components of Occupant Impact Velocity (OIV) and 2) the maximum lateral and longitudinal component of vehicle acceleration averaged over 10 millisecond intervals after occupant impact, i.e., the occupant ridedown acceleration (ORA). Also shown in the table are the EN 1317 occupant risk factors, which include the Theoretical Head Impact Velocity (THIV), the Post Impact Head Deceleration (PHD) and the Acceleration Severity Index (ASI).

Table 13.	Summary	of occupant	risk measures	computed from	test 7069-37	and FEA	simulation

Occupant Risk Fac	tors	Test 7069-37	FEA (ORIL_151203)	Preferred Limits
Occupant Impact Velocity	x-direction	1.4	3.3	9
(m/s)	y-direction	-2.9	-1.8	9
THIV (m/s)		3.3	3.3	9

Ridedown Acceleration	x-direction	-4.4	-2.6	15
(g's)	y-direction	6.3	-6.5	15
PHD (g's)		6.4	6.8	15
ASI		0.42	0.33	
Max 50-ms moving avg. acc.	x-direction	-1.4	-2.4	
(g's)	y-direction	-3.8	2.7	
	z-direction	-2	-1.7	
Maximum Angular Disp.	Roll	16.2	26.2	
(deg)	Pitch	3.3	-4	
	Yaw	-16.7	-17.3	

The results indicate that the occupant risk factors for both the full-scale test and the simulation are of similar magnitudes and are both well below the preferred limits in NCHRP Report 350. The occupant impact velocity in the longitudinal direction was predicted from the simulation to be 3.3-m/s (1.9 m/s higher than the test OIV of 1.4 m/s). In the transverse direction, the occupant impact velocity predicted in the simulation was 1.8-m/s (1.1 m/s lower than the test OIV of 2.9 m/s). All values were well below the preferred limit of 9 m/s. The highest 0.010-second occupant ridedown acceleration in the longitudinal direction was 2.6 g (1.8 g higher than the test ORA of 4.4 g). In the transverse direction, the highest 0.010-second occupant ridedown acceleration was 6.5 g (0.2 g higher than the test ORA of 6.3). Again, all were well below the preferred limit of 15 g's. In the longitudinal direction, the maximum 50-ms moving average acceleration value computed in the analysis was 2.4 g's compared to 1.4 g's measured in the full-scale test. In the transverse direction, the maximum value computed in the analysis was 2.7 g's compared to 3.8 g's measured in the full-scale test.

The comparison of the CEN metrics is also shown in Table 13 which show very good agreement between the test and analysis. For example, the difference between the FEA and test for the THIV, PHD and ASI was 0 percent, 6.25 percent and 21 percent, respectively.

Analysis Summary

The response of the modified Illinois Two-Tube bridge rail was evaluated under NCHRP Report 350 Test 4-12 impact conditions. The primary purpose for the analysis was to check for potential issues for the modified mounting system in a simulated crash condition, and to ensure that all loading conditions for the post and mount have been addressed in the study. For example, it was important to determine how the modified system would respond when the cargo-box impacted the top of the rail and snagged on the posts, as well as how the mount would respond to the combined vertical, lateral and longitudinal load.

The results of the analysis were also compared to the results of the full-scale test that was performed on the baseline system. Because of differences in the way the cargo-box was fastened to the truck frame in the FE model compared to that of the test vehicle, it was not possible to make a direct comparison of the time-history data. In particular, the cargo-box slid off the truck frame during the full-scale test which increased the roll angle of the box. This event affected the overall dynamics of the vehicle, but it also affected the measured accelerations which were

collected from accelerometers mounted directly underneath the cargo-box. In the model, the front of the cargo-box was rigidly constrained to the truck frame thus causing the cargo-box and the truck to move and rotate more as a single unit. No attempt was made to revise the vehicle model to better match that of the test vehicle, since that effort was not in the scope of the project. The primary purpose was to ensure that the basic behavior of the bridge rail (e.g., barrier deformations, potential for snags, etc.) was consistent with the baseline system and to ensure that the loads transmitted to the bridge superstructure under NCHRP Report 350 Test 4-12 impact conditions did not cause damage to the bridge.

Comparison of the sequential views for the analysis and test of Test 4-12 impact conditions indicated that the response of the modified barrier, as well as the basic kinematic behavior of the vehicle, was very similar to that of the baseline system. The analysis indicated that the modified system would safely contain and redirect the 18,000-lb truck impacting at 50 mph and 15 degrees. The damage to the posts and railing was minor with only 1 inch of permanent deflection. The damage to the bridge superstructure was also considered minor, with minimal plastic deformation of the bridge components. The tensile forces in the 7/8-inch A325-1 bolts that connect the mounting tube, fascia beam and tee connector were of adequate strength to prevent failure of the connection (i.e., to prevent disconnection of the diaphragm and fascia beam).

MASH TL3 Crash Simulations

It was recently announced by AASHTO's Technical Committee on Roadside Safety (TCRS) that by the year 2019 all bridge rails installed on the national highway system must meet *MASH* TL3 conditions. It is uncertain how this will affect the eligibility of the modified Illinois Two-Tube bridge rail, since this system is only intended to be used on Ohio's local road system. It was therefore decided to further evaluate the performance of the bridge rail with modified post mount under MASH TL3 impact conditions using finite element analysis. The results of the analyses not only provide additional crash performance information, but also provide ODOT with the expected outcome should ODOT elect to perform full-scale testing on the system at a future date. MASH TL4 conditions (i.e., 22,000-lb single unit truck impacting at 56 mph and 15 degrees) were not considered in these analyses since it was expected that the height of the barrier (i.e., 32 inches) would not be sufficient to prevent the single unit truck from rolling over the bridge rail during impact.

The analyses were performed under worse-case conditions for the bridge structure, which corresponded to the lowest stiffness for the bridge superstructure, as well as for the post-mount design options. The minimum bridge superstructure size listed as Case A in Table 6 was selected for this evaluation, which included the W14x30 fascia and stringer beams with C12x25 diaphragms, and WT6x36 Tee connectors. The post stiffener Design B (see Figure 5) was also included in this evaluation with the length of the post set at maximum length corresponding to a maximum bridge deck thickness of 8 inches.

MASH requires two tests for assessing the crash performance of longitudinal barriers: Test 3-10 (i.e., 2,420-lb sedan impacting at 62 mph and 25 degrees) and Test 3-11 (i.e., 5,000-lb quad-cab ¹/₂-ton pickup impacting at 62 mph and 25 degrees). The vehicle models used for these analyses were the YarisC_v1L model (based on a 2010 Toyota Yaris) and the SilveradoC_v3a model (based on a 2007 quad-cab Chevy Silverado). These vehicles closely represent the two test vehicles specified in MASH. [AASHTO09] The vehicle models were developed through the process of reverse engineering by the members of George Mason University (GMU) and were initially validated based on NCAP frontal wall impact tests through comparison with NHTSA test data. The models also include validated suspension and steering subsystems. The Silverado model has been continually improved by GMU, as well as the user community, since its development and has been used successfully in several studies involving crash analysis with roadside safety hardware. The Yaris model has not been exercised as much by the user community as the Silverado, but was expected to provide reasonable results. Figure 76 shows an example of the Silverado model impacting against the ODOT 50-inch portable concrete barrier. The latest release of this model is the SilveradoC_v3a which was obtained from GMU in March 2016.



concrete barrier [Plaxico06]

Time = 0.1 seconds

Time = 0.4 seconds

Time = 0.67 seconds Figure 76. Impact analysis of the Silverado model impacting against the ODOT 50-inch portable



Figure 77. GMU Yaris model

The model setup and boundary conditions were the same as those used for the SUT test in the previous section. The general layout and extent of the FEA model is shown in Figure 78. The impact speed and angle for both cases were 62.1 mph and 25 degrees, respectively. The impact point was 3.6 feet upstream of the critical post for Test 3-10 and 4.3 feet upstream of the critical post for Test 3-11, as specified in MASH (refer to Table 2-6 in MASH).





Figure 78. Overall model setup for the MASH Test 3-10 and Test 3-11 analyses.

The Test 3-10 simulation analysis was performed for 0.6 seconds of the crash event, and the Test 3-11 simulation analysis was performed for 0.8 seconds of the crash event. The analyses were conducted using a time-step of 1.0 microsecond. The performance of the bridge rail was evaluated based on structural adequacy, vehicle stability during and after redirection and occupant risk. MASH requires that the bridge rail must redirect the vehicle without allowing the vehicle to penetrate the system, the vehicle must remain upright during and after redirection, occupant impact with the interior of the vehicle must not exceed velocities of 40 ft/s (12.2 m/s) and the longitudinal ride-down accelerations of the occupant must not exceed 20.49 g's. Data from the accelerometer located at the center of gravity of the vehicle model was collected and entered into TRAP to calculate standardized occupant risk factors. The results of the analysis were also critically evaluated to identify any deficiencies in the various components of the system that may affect crash performance or cause unacceptable damage to the bridge structure.

Sequential Views and Time-History Plots

Sequential views of the analysis from an overhead and downstream view point are shown in Figures 79 and 80 for Test 3-10 and Test 3-11, respectively. The acceleration-time history plots from the analyses are shown in Figure 81. These include the 10-millisecond moving average, which is used to determine occupant ridedown accelerations; and the 50-milliseond moving average accelerations. The angular rate- and displacement-time history plots are shown in Figure 82. The sequential views for both Test 3-10 and Test 3-11 showed smooth redirection of the vehicle from the barrier with minimal potential for snagging. The results also showed that the small car remained very stable with minimal roll or pitch of the vehicle throughout the analysis. The peak roll and pitch angle of the car was 7.3 degrees and 2.7 degrees, respectively. The pickup, on the other hand, remained stable but showed relatively high roll angle after exiting from the bridge rail during redirection. The maximum roll for the pickup was 38.8 degrees and was decreasing at the termination of the analysis at 0.8 seconds. The maximum pitch for the pickup was 8.4 degrees. These values were well below the critical limits of 75 degrees for both roll and pitch as specified in *MASH*.

Occupant Risk Metrics

Table 14 shows the occupant risk measures calculated from the results of the FEA analyses. The acceleration data was pre-filtered using an SAE Class 180 filter before input into TRAP. The table also includes the preferred and limiting values, according to MASH, for the lateral and longitudinal components of Occupant Impact Velocity (OIV) and the maximum lateral and longitudinal occupant ridedown acceleration (ORA). Also shown in the table are the EN1317

occupant risk factors, which include the Theoretical Head Impact Velocity (THIV), the Post Impact Head Deceleration (PHD) and the Acceleration Severity Index (ASI).

For Test 3-10 the occupant impact velocity in the longitudinal and lateral directions were 23.95 ft/s (7.3 m/s) and 32.81 ft/s (10.0 m/s), respectively. The lateral OIV value exceeded the preferred limit of 30 ft/s (9.1 m/s), but was below the critical limit of 40 ft/s (12.2 m/s) specified in MASH. The highest 0.010-second occupant ridedown acceleration values in the longitudinal and lateral directions were 8.8 g and 8 g, respectively; which were also within preferred limits of 15g in MASH. The values of CEN metrics are included in the table for completeness, although they are not part of the evaluation criteria in MASH.

For Test 3-11 the occupant impact velocity in the longitudinal direction was 19.36 ft/s (5.9 m/s) and 24.28 ft/s (7.4 m/s), respectively. The highest 0.010-second occupant ridedown acceleration values in the longitudinal and lateral directions were 6.6 g and 11.3 g, respectively. All occupant risk metrics for this case were within preferred the limits specified MASH. Again, the CEN metrics were included for completeness.

Damage to Bridge Rail

The damage to the bridge rail was moderate for both test cases, as shown in Figures 83 through 88. For Test 3-10, the dynamic deflection was 5.11 inches and the maximum permanent deflection was 1.20 inch. For Test 3-11, the dynamic deflection was 7.16 inches and the maximum permanent deflection was 3.42 inches. Figure 88 shows the damages to the post mount for both cases which was primarily limited to deformation of the front mounting plates. The deformation resulted in a permanent gap-separation between the plates of 0.24 inches and 0.52 inches for Test 3-10 and Test 3-11, respectively. There was no failure of the mount connections.



Figure 79. Sequential views for the FEA simulation of MASH Test 3-10 from overhead and downstream viewpoints.



Figure 80. Sequential views for the FEA simulation of MASH Test 3-11 from overhead and downstream viewpoints.



Figure 80. [Continued] Sequential views for the FEA simulation of MASH Test 3-11 from overhead and downstream viewpoints.



Figure 81. 10-millisecond and 50-millisecond moving average acceleration-time history plots collected at the c.g. of the vehicle for Test 3-10 and Test 3-11 analyses.



Figure 82. Angular rate- and displacement-time history plots collected at the c.g. of the vehicle for Test 3-10 and Test 3-11 analyses.

Occupant Risk Fact		MASH TL-3				
	//5	Test 3-10	Test 3-11			
Occupant Impact Velocity	x-direction	7.3	5.9			
(m/s)	y-direction	10	7.4			
	at time	at 0.0736 seconds on right side of interior	at 0.0929 seconds on right side of interior			
THIV		12.1	9.3			
(m/s)		at 0.0736 seconds on right side of interior	at 0.0929 seconds on right side of interior			
Ridedown Acceleration	v-direction	-8.8	-6.6			
(g's)	x-unection	(0.0754 - 0.0854 seconds)	(0.0956 - 0.1056 seconds)			
	v-direction		-11.3			
y-unection		(0.0771 - 0.0871 seconds)	(0.2153 - 0.2253 seconds)			
PHD		13	11.4			
(g's)		(0.0736 - 0.0836 seconds)	(0.2152 - 0.2252 seconds)			
451		2.25	1.49			
		(0.0292 - 0.0792 seconds)	(0.0319 - 0.0819 seconds)			
Max 50-ms moving avg. acc.	v-direction	-13.7	-9.3			
(g's)	x-unection	(0.0161 - 0.0661 seconds)	(0.0352 - 0.0852 seconds)			
	v direction	-17.4	-11.5			
	y-unection	(0.0241 - 0.0741 seconds)	(0.0318 - 0.0818 seconds)			
			-3.7			
	2-unection	(0.0497 - 0.0997 seconds)	(0.0438 - 0.0938 seconds)			
Maximum Angular Disp.		7.3	38.8			
(deg)	Roll	(0.3021 seconds)	(0.5595 seconds)			
Pitch Yaw		-2.7	-8.4			
		(0.1891 seconds)	(0.6954 seconds)			
		-48.2	-35.1			
		(0.7893 seconds)	(0.5636 seconds)			

 Table 14.
 Summary of occupant risk measures computed from Test 3-10 and Test 3-11 FEA results.



Figure 83. Damage to bridge rail in FEA simulation of MASH Test 3-10 and Test 3-11.



Figure 84. Maximum dynamic deflection of bridge rail during FEA of MASH Test 3-10.



Figure 85. Maximum permanent deflection of bridge rail during FEA of MASH Test 3-10.

Max Dynamic Deflection = 7.16" @ 0.075 seconds



Figure 86. Maximum dynamic deflection of bridge rail during FEA of MASH Test 3-11.

Max Permanent Deflection = 3.42" @ 0.765 seconds



Figure 87. Maximum permanent deflection of bridge rail during FEA of MASH Test 3-11.



Figure 88. Deformations at the top of two post mounts resulting from the cargo-box snagging on the posts.

Damage to the Bridge Structure

Figure 89 shows sequential views of the deformations of the bridge components at the location of maximum loading (i.e., at Post 2) from an overhead viewpoint for both the Test 3-10 and Test 3-11 analysis cases. The flanges of the fascia beam and the diaphragm were made transparent to facilitate viewing. For the Test 3-10 case the maximum dynamic separation between the mounting plate and the fascia beam was 0.35 inches at 0.065 seconds of the impact event. The maximum permanent separation was 0.06 inches ($\approx 1/16$ inch). For the Test 3-11 case the maximum dynamic separation between the mounting plate and the fascia beam the mounting plate and the fascia beam was 0.215 seconds of the impact event. The maximum permanent separation between the mounting plate and the fascia beam was 0.215 networks of the impact event. The maximum permanent separation was 0.12 inches ($\approx 1/8$ inch).



Figure 89. Sequential overhead views of maximum dynamic and permanent deformation of bridge components at location of maximum loading for (a) MASH T3-10 and (b) MASH T3-11.

Figure 90 shows contours of effective plastic strain in the bridge components at the time of maximum loading for both the Test 3-10 and the Test 3-11 cases. The hardware was removed from view in order to show the maximum strains around the bolt holes in the tee connector, the diaphragm and the fascia beam. The maximum strain in both cases occurred at the edge of the bolt holes in the diaphragm and Tee connector elements. For the Test 3-10 case the maximum strain in the diaphragm and the web of the tee connector was 0.032. The maximum strain in the flange of the tee was 0.012, and the maximum strain for the fascia beam was 0.004. These values are considered negligible and would thus not require repair or replacement after impacts with similar or less severity.

For the Test 3-11 case the maximum strain in the diaphragm element and web of the tee connector element was 0.073 and 0.068, respectively. These values are considered moderate and may require repair after an impact with this level of crash severity. The maximum strain in the flange of the tee element was 0.036, and the maximum strain for the fascia beam was 0.04. These values are considered negligible and would thus not require repair. The forces in the bolted connections in the fascia beam reached a peak value of 33 kips, which corresponds to a maximum stress of 71 ksi for a 7/8-inch diameter UNC bolt. Recall that the yield stress for A325-1 steel is 92 ksi.



Figure 90. Contours of effective strain for bridge components (with bolts removed from view) at location of maximum loading.

Guardrail-to-Bridge Rail Transition Section

Introduction and Background

When two barrier systems of different stiffness are connected together, such as connecting a semi-rigid guardrail to a rigid bridge rail, it is necessary to ensure a gradual transition across the connection point. Any abrupt change in stiffness of the barrier can lead to pocketing, snagging and/or penetration of the barrier during impact. Thus, a transition guardrail section is necessary to develop gradual stiffness between the two barrier systems. Developing a transition system for use with the modified Illinois Two-Tube bridge rail was beyond the scope of this study. The purpose of this chapter is to present a review of existing transition system designs and to make recommendations for the most appropriate system to be used with the modified Illinois Two-Tube bridge rail. Bridge rail transitions from three different state inventories, including Oregon, Illinois and Ohio, are presented.

Transitions Currently Used with the Illinois Two-Tube Bridge Rail

The Illinois Two-Tube bridge rail (baseline system) is currently used in the states of Illinois and Oregon, thus the transition system(s) used with this bridge rail in those states should also be applicable to the modified bridge rail design. The Oregon Two-Tube bridge rail is essentially equivalent to the Illinois design, with only minor differences. For example, the top-of-rail height is a ¹/₂ inch taller for Oregon design (i.e., 32.5 inches) compared to the Illinois design (32 inches);

and the Oregon design includes a 1.5-inch chamfer at the top of the post, which is considered inconsequential to the design. The transition systems used with this bridge rail in Oregon and Illinois, however, are significantly different. The Oregon transition is shown in Figures 91 through 93 and the Illinois transition is shown in Figures 94 through 97. Both the Oregon and Illinois transition systems are classified as NCHRP Report 350 Test Level 3 (TL3) systems.

Oregon Transition Design

The Oregon transition system is similar to a transition system crash tested at TTI (i.e., Test 401021-3) under NCHRP Report 350 Test 3-21 impact conditions. The test involved a 4400-lb pickup impacting the thrie-beam transition at 82 inches upstream of the first bridge rail post. The impact speed was 62.1 mph and the impact angle was 25 degrees. The system successfully met the crash test evaluation criteria of NCHRP Report 350.[*Alberson05*]

The standard drawings for the Oregon transition system are provided in Appendix O. This system uses a 0.135-inch thick thrie-beam terminal connector that attaches to a guardrail connector plate assembly, which in turn attaches to the two-tube rails, as shown in Figures 91 through 93. The top of the thrie-beam terminal connector is mounted 1 inch below the top of the top bridge rail resulting in a top-of-rail height of 31.5 inches. The thrie-beam terminal connector is attached to 12.5-ft section of thrie-beam rail composed of two layers of nested 12-gauge thriebeam. The end of this section is then attached to two layers of nested 12-gauge thrie-beam-to-wbeam transition elements with a symmetrical flare. The top of the rail height at the w-beam end is 27 ¹/₂ to 27 ³/₄ inches which allows for connection to a standard-height G4(1S) or G4(2W) wbeam guardrail system. The first post of the transition is located 27.5 inches from the end-post of the bridge rail. The next five posts are all spaced at 18.75 inches (1/4 post spacing); the next three posts are spaced at 37.5 inches (1/2 post spacing). The Oregon drawings indicate that either the steel W6x9 post or wood 8"x8" post can be used for the transition system. The embedment depth of the posts was not provided on the standard drawings; however, the crash test report for this transition system shows the embedment depth for the wood posts to be 48 inches (min). The blockouts are 8"x8" wood blocks.



Figure 91. Oregon Two-Tube Transition, Elevation View [ORDOT01]



Figure 92. Oregon Two-Tube Transition, Connection View [ORDOT01]



Figure 93. Oregon Two-Tube Transition, Section B-B [ORDOT01]

Illinois Transition Design

The transition system used in Illinois is called "Traffic Barrier Terminal, Type 6A", which is shown in Figures 94 - 97. This transition system is designed to connect the bridge rail to a 31-inch tall w-beam guardrail. Like the Oregon design, the bridge rail approach section of this transition system design is also similar to that tested by TTI in test 401021-3.[*Alberson05*]

The Illinois design uses a modified thrie-beam end-shoe attached to a guardrail connection plate. The connection plate is larger than the one used in the Oregon design and there are two design options depending on whether it is being used on a curb-mounted or side-mounted bridge rail system. The system uses an asymmetrical thrie-beam to w-beam transition with the flare on the bottom side of the rail section. The top of rail height is 31 inches throughout the transition and allows for connection to 31-inch high guardrail, such as the MGS system. On the bridge rail end of the transition, the first section of rail is composed of two layers of nested 12-gauge thrie-beam supported by W6x9 steel posts with 6"x8"x18" rectangular wooden blockouts spaced at 18.75 inches on center (e.g., quarter post-spacing). The next section of rail is composed of a single layer 12-gauge thrie-beam supported by W6x9 steel posts with 6"x8"x18" blockouts spaced at 37.5 inches on center (e.g., half post-spacing). The next section

of rail is composed of a single layer 10-gauge thrie-beam-to-w-beam transition section supported by W6x9 steel posts at half post-spacing. The thrie-beam end of the transition uses a 6"x8"x18" blockout, the midpoint of the transition uses a 6"x12"x19" blockout, and the w-beam end of the transition uses a 6"x12"x14" blockout. The final section of rail is composed of a single layer of 12-gauge w-beam supported by W6x9 steel posts with 6"x12"x14" blockouts at half postspacing. The embedment depth for Posts 1 through 15 is 59 inches and the embedment depth for Posts 16 and 17 is 41 inches. The overall length of the system is 43.75 feet.

Note that the standard drawings for this system show it connected to a curb-mounted bridge rail (see Appendix P), but in the General Notes section of the drawings it is stated that the attachment to the side mounted rail is similar.



Figure 94. Illinois Type 6A Traffic Barrier Terminal, Elevation and Plan Views. [ILDOT02]



Figure 95. Illinois Type 6A Traffic Barrier Terminal, Sections A-A and B-B. [ILDOT02]



Figure 96. Illinois Type 6A Traffic Barrier Terminal, Section C-C. [ILDOT02]



Figure 97. Illinois Type 6A Traffic Barrier Terminal, Transition Section. [ILDOT02]

The asymmetrical w-beam to thrie-beam transition design for the Illinois system is the same as the design tested at the Midwest Roadside Safety Facility in Tests MWT-5 and MWT-6, which is shown in Figure 98. Test MWT-5 involved a 4,431-lb pickup truck impacting the guardrail system upstream from the w-beam to thrie-beam transition element at a speed of 61.5 mph and at an angle of 24.9 degrees. Test MWT-6 involved a 1,992-lb small car impacting the guardrail system upstream from the w-beam to thrie-beam transition element at a speed of 65.5 mph and at an angle of 20.4 degrees. In both tests, the vehicles were successfully contained and redirected. The test summary sheets for Tests MWT-5 and MWT-6 are provided in Appendix P.

The details of the crash test and results are documented in MwRSF Report TRP-03-167-07. [*MwRSF01*] The FHWA Eligibility Letter for this system is B-187. It should be noted that there is a discrepancy between the crash report and the eligibility letter. The drawing in the report shows that the wooden blockout at Post #11, which is the post attached to the asymmetrical 10-gauge thrie-beam to w-beam transition, was 6"x12"x14 ¼", while the description in the FHWA Eligibility letter showed the blockout sized at 6"x8"x14 ¼".



Figure 98. Transition System Crash Tested in Report TRP-03-167-07. [MwRSF01]

Ohio Transition Design

Ohio currently has an NCHRP Report 350 TL3 compliant transition design that is used for transitioning the MGS guardrail to the Ohio Twin Steel-Tube (TST) bridge railing, as well as concrete parapet systems. This system is listed under the Ohio standards as the MGS Bridge Terminal Assembly Type 1 (or MGS-3.1). The standard drawings for this system are provided in Appendix Q. Figure 99 shows the MGS-3.1 connecting to a concrete parapet, but a note on Page 2 of the standard drawings also includes connection details for attachment to the TST bridge railing, as illustrated in Figure 100.



Figure 99. Ohio MGS Bridge Terminal Assembly, Type 1. [OHDOT01]

Like the previous designs, the W6x9 steel post or the 6"x8" wood post may be used, but the same post material should be used throughout the length of the transition. The first section of the transition (from the bridge rail end) is $12 \frac{1}{2}$ feet long and composed of two layers of nested 12-gauge thrie-beam; the embedment depth for the first six posts is 49 inches for the steel posts or 52 inches for the wood posts. The remaining two posts in this section have an embedment depth of 40 inches for both the steel and wood post options. The next section of the transition is 6.25 inches long and composed of a single layer of 12-gauge thrie-beam; the embedment depth for both the steel and wood post options is 40 inches. The final section of the transition is composed of an asymmetrical 10-guage thrie-beam-to-w-beam transition element. The system can use either 6"x12"x19" or 6"x12"x22" wood blockouts from the bridge rail connection up to the midpoint of the asymmetrical transition panel. The w-beam end of the asymmetrical transition panel has a standard MGS 6"x12"x14" blockout. At this point, the transition connects to the Midwest Guardrail System at a height of 31 inches. Figure 99 also shows the transition with a curb, but a note on the drawings states that the curb is not required when attaching to the TST bridge rail.

The Ohio design is very similar to TRP-03-167-07¹, which was successfully crash tested at the w-beam to thrie-beam transition section. Both systems are the same length and have the same rail components. Both have a 12 ½-foot nested thrie-beam section that attaches to a single layer 6 foot 3 inch thrie-beam section that is in turn attached to a 10-gauge asymmetrical thrie-beam section, and both systems connect to an MGS guardrail system. The primary difference between these two systems is that they use different post spacing at the approach to the bridge rail. The TRP-03-167-01 system uses half post spacing throughout; whereas the Ohio design, like the Oregon and Illinois designs, uses quarter post spacing at the bridge rail approach.

Discussion

Bridge rail transitions from three different state inventories, including Oregon, Illinois and Ohio, were reviewed. A visual comparison of these designs, along with two related crash tested designs (i.e., Test 401021-3 and TRP-03-167-07), is shown in Figure 101. Each system is unique and are all considered eligible for use on Federal-Aid projects as NCHRP Report 350 TL3 systems. The Two-Tube Side Mount Rail Transition used by Oregon DOT was the only transition system that attaches to a 27 ³/₄ inch high guardrail system. It is also the shortest system and consists of nested thrie-beam rail throughout the entire length of the system. The Traffic Barrier Terminal, Type 6A, which is used by Illinois DOT, is the longest system, as it uses a 12 ¹/₂ foot single layer thrie-beam panel where both the Ohio and Report TRP-03-167-07 systems use a 6.25-foot panel for the single-layer thrie-beam section.

¹ It should also be noted that TRP-03-167-07 was tested with steel posts, and there was no mention in the report or in Acceptance Letter B-187 that the system may be used with wooden posts.



Figure 100. Ohio MGS Bridge Terminal Assembly Steel Tube Connection. [OHDOT01]

The transition designs at the approach section to the bridge rail were the same for Oregon, Illinois and Ohio. This design was full-scale crash tested by TTI in Test No. 402021-3 and, therefore, all systems meet NCHRP Report 350 TL3 for this section of the transition at the bridge rail approach based on the results of that test. The Illinois and the Ohio designs both use an asymmetrical 10-gauge thrie-beam-to-w-beam transition element, which connects to a 31-inch tall guardrail. This design was successfully crash tested by MwRSF in Tests MWT-5 and MWT-6; it is therefore considered NCHRP Report 350 TL3 compliant for the guardrail-end of transition section based on the results of those tests. The Oregon design uses two layers of 12gauge sections, rather than the single layer of 10-gauge; however, this option is also considered to be of equivalent performance as the tested system.

It is the author's opinion that any of the three transitions designs (i.e., Oregon, Illinois or Ohio) may be used with the modified Illinois Two-Tube bridge rail system. These systems all meet NCHRP Report 350 TL3 and are therefore eligible for use on Federal-Aid projects. The

Ohio DOT should, however, perform an independent review of these designs to confirm this assessment.

The Oregon and Illinois transition designs can be used as detailed in the standard drawings from the respective states, which are shown in Appendices O and P, respectively. For attachment to the Ohio MGS-3.1 transition, however, a new connection detail was developed based largely on the Oregon standard. In particular, the rail caps and Connection Angle "A" from the Oregon design was adopted for connecting the thrie-beam terminal connection plate on the bridge-rail end of the Ohio MGS-3.1 transition to the bridge rail. A primary difference between the new design and that of Oregon is that the ends of the tubular rails were shortened from 2'-2 ¹/4" to 2'-0 5/16" (i.e., the distance from the center line of the first bridge rail post to the ends of the rail elements. The detailed drawings for the connection of the Ohio MGS-3.1 to the Modified Illinois Two-Tube bridge rail are included with the bridge rail drawings in Appendix B.

For connection to the 27.75-inch tall guardrail, the Oregon design appears to be the best option. An FHWA Eligibility letter for the thrie-beam-to-w-beam transition section for that system has not yet been identified; however, this system is currently used in Oregon as an NCHRP Report 350 TL3 system and is specifically used for connection to the Illinois two-tube bridge rail. The Ohio DOT will need to review the system details to confirm that they meet Ohio standards.



Figure 101. Comparison of Available and Tested Transition Systems.

SUMMARY AND CONCLUSIONS

The Illinois Two-Tube bridge rail was modified in this study to accommodate attachment of the system to steel bridge fascia beams. The original (baseline) system includes a side mount design that fastens directly to the side of a concrete bridge deck. Detailed drawings for the baseline design are shown in Appendix A. This system was crash tested at TTI in 1993 under crash test numbers 7069-35 (small car test), 7069-36 (pickup test) and 7069-37 (SUT test). The modifications that were made to the bridge rail in the current study were limited to the postmount design. Since there were no changes to the bridge rail design for the components above the road surface, the modifications were therefore considered to be "non-significant," and engineering analysis was used to evaluate the design changes.

The analysis was carried out using a combination of engineering calculations, pendulum testing and finite element analysis. The basic approach was to (1) determine the force-deflection response (stiffness) of the post for the baseline deck-mounted design, (2) develop a fascia mount design that provides equivalent stiffness to the baseline design, (3) verify the stiffness response for the new design, (4) ensure that loads transferred onto the bridge fascia beam do not result in excessive damage to the bridge superstructure, (5) perform simulation of NCHRP Report 350

Test 4-12 on the new design to verify that the modifications do not adversely affect crash performance and (6) perform MASH TL3 simulations to further evaluate the performance of the system under the new crash testing procedures.

Two fascia-mount designs were developed and the detailed drawings are included in Appendix B. The new fascia mount designs include two different post-stiffener options, and both options provide adequate strength for developing the required stiffness response for the post. Alternatively, the target stiffness could be achieved by selecting a larger wide-flange section with appropriate strength properties for the post. The decision to add the *stiffeners* on the post was made (based on conversations with the FHWA Office of Safety) to ensure that the original design for the bridge rail system was maintained above deck level.

The designs for steel bridges vary, thus another important part of the study involved determining the minimum size requirements for the bridge components for each mounting design option. The post-and-mount design specifications, as well as the minimum size requirements for the bridge diaphragm and Tee connector elements for various fascia beam sizes were presented in Table 10. However, to further limit the number of design variations and to further reduce the potential for damage to the bridge superstructure, it is recommended that the minimum size for the diaphragm and tee connector elements be specified as C12x25 and WT6x36, respectively, for all applicable fascia beam options. It is also recommended that post stiffener Design B be used for all cases. Design B was shown via pendulum testing to have essentially equivalent stiffness as Design A for post deflections of up to 9 inches. Design B also resulted in lower stress concentrations in the weld located at the top of the 1-inch thick post-mounting-plate (between the mounting-plate and the post)². Table 15 shows the recommended minimum size specifications $\frac{1}{2}$ for the bridge diaphragm and Tee connector elements and the appropriate mounting design options for fascia beams of size W14x30 and larger. Note that the bridge components listed in Table 15 are minimums, thus larger structural sections may be used. However, the W14x30 is the smallest structural section that can be used for the fascia beam in order to maintain adequate stiffness of the bridge rail without incurring excessive damage to the bridge superstructure.

Fascia Beam			Diaphragm		WT Connector		Mounting Tube			Post-		
	Min. Thickness			Min. Thickness			Min. Thickness			Thicknoss	Longth Stiffonor	
Size	Flange	Web	Size	Flange	Web	Size	Flange	Web	Size	(in)	(in)	Docign
	(in)	(in)		(in)	(in)		(in)	(in)				Design
W14x30 and larger	0.385	0.27	C12v25	0 501	0 297	W/T6v26	0.67	0.42	TS12v6v0 25	0.25	15	В
W14 sections	0.385	0.505 0.27	C12725	0.501	0.301 0.387	VV10x30	0.07	0.45	1312X0X0.23	0.25	15	(horz. plates)
W16x40 and larger												В
section sizes	0.505	0.305	C12x25	0.501	0.387	WT6x36	0.67	0.43	TS14x6x0.25	0.25	15	(horz. plates)

 Table 15. Recommended minimum size specifications for bridge components and corresponding post-mount design (for implementation).

The post-mount should be fastened onto the web of bridge fascia beam such that the top edge of the mount plate contacts the bottom edge of the radius at the junction of the web and top flange of the fascia beam. This positioning is necessary to minimize the length of the post and maintain proper stiffness of the post-mount system (refer to the drawings in Appendix B). Also,

 $^{^2}$ From the FEA, stress concentrations for Design A were identified at the outside edges of the post flange where the stiffeners were positioned. This was also the point of failure initiation for the weld in Test 15009G.

the length of the mounting tube should not exceed 15 inches due to the potential for excessive vertical deflection of the post as the post-mount rotates under lateral impact loads.

A strength analysis was performed using FEA in which a lateral load was applied to an individual post that was mounted onto the bridge fascia beam. The results showed that deformations of the bridge components would remain below critical levels as long as the lateral deflection of the bridge rail post remained below 12 inches. The results of the analysis were used to determine the appropriate size for the mounting bolts and the welds on the post-mount to ensure that the post-mount provided adequate strength for TL4 impact loading but would fail prior to excessive loads being transferred onto the bridge superstructure. The target strength was 27 kips and the target failure deflection range was 8 to 11 inches. The final design included two 1-inch diameter A325-1 bolts at the top-mounting location at the post-side of the mount and a 5/8-inch fillet weld between the post-flange and the 1-inch thick mounting plate. Physical pendulum testing was then performed which confirmed the strength performance of the post-mount design.

FEA was then used to simulate NCHRP Report 350 Test 4-12 for the modified bridge rail system. The results of that analysis indicated that the modified mount design provided equivalent or greater overall stiffness for the posts compared to the original mount design. The maximum lateral deflection of the post under NCHRP Test 4-12 impact conditions was approximately 4.3 inches. The loads transferred into the bridge superstructure resulted in relatively low plastic deformations of the bridge components and were considered to be negligible. The results further indicated that the design modifications met all structural capacity, occupant risk measures and vehicle stability criteria set forth in NCHRP Report 350. The Illinois Two-Tube bridge system is currently rated as a Report 350 Test Level 4 barrier. Based on the analysis performed herein, the modified post-mount design provided equal or greater stiffness for the bridge rail system under NCHRP Report 350 TL4 impact conditions, and thus should result in equivalent or better crash performance for the system. The design modifications are further considered to be non-significant, regarding the changes to the original Report 350 TL4 design, since the effects of the changes were shown to be inconsequential to the performance of the system with respect to the baseline design.

The performance of the bridge rail with modified post mount was also evaluated under MASH TL3 impact conditions using finite element analysis. Two analysis cases were performed per MASH requirements for longitudinal barriers. These included Test 3-10 (i.e., 2,400-lb sedan impacting at 62 mph and 25 degrees) and Test 3-11 (5,000-lb quad-cab pickup impacting at 62 mph and 25 degrees). The analyses were performed under worse-case conditions for the bridge structure, which corresponded to the lowest stiffness for the bridge superstructure, as well as for the post-mount design options. Both tests met all structural capacity, occupant risk and vehicle stability criteria set forth in MASH. The bridge rail safely contained and redirected the vehicle in both analysis cases with only moderate damage to the bridge rail system. The small car analysis case resulted in very stable redirection; while the pickup analysis case resulted in a relatively high roll angle (i.e., 38.8 degrees), but still well within the MASH limit of 75 degrees. The occupant impact velocities and the occupant ridedown accelerations were well below critical limits for both cases. The maximum permanent deflection of the rail was 1.2 inch and 3.4 inches, for Test 3-10 and Test 3-11, respectively. The damage to the bridge structure was limited primarily to the edges of the top bolt hole at the connection between the diaphragm and Tee connector elements. The damages for the Test 3-10 analysis case was considered negligible with effective plastic strain values less than 0.05; while the damages for the Test 3-11 analysis case were considered moderate with effective strain values of approximately 0.08. Thus, repair of one or more of the diaphragm and Tee connector elements may be required after impacts with an impact severity equal to or greater than that of Test 3-11.

Several options for NCHRP Report 350 TL3 compliant transitions systems were identified for use with the bridge rail. These include the Oregon 2-*Tube Side Mount Rail Transition*, the Illinois *Traffic Barrier Terminal Type 6*, and the Ohio *MGS Bridge Terminal Assembly Type 1 (MGS-3.1)*. Refer to Appendices O, P and Q for the drawing details. The Illinois and the Oregon designs are currently being used with the Illinois Two-Tube bridge rail system (original bridge rail design) in their respective states. The Ohio design is essentially equivalent to both the Illinois and Oregon designs regarding the transition section leading up to the bridge rail. Thus, it is also considered to be applicable for use with this bridge rail system. The connection details for connecting the bridge rail to the Ohio MGS-3.1 transition are provide in the bridge rail drawings in Appendix B.

RECOMMENDATIONS FOR IMPLEMENTATION OF RESEARCH FINDINGS

The Illinois two-tube bridge rail with the fascia-mount design may be implemented in the field as an NCHRP Report 350 TL4 system; however, it is recommended by the research team that the system only be installed on the local road system and only be used as an NCHRP Report 350 TL3 device. This system is eligible for use on federal-aid reimbursement projects and may be installed on steel bridges with fascia beams of size W14x30 or larger. The mount should not to be used with other fascia beam types without additional analysis and testing to verify crash performance.

It was recently announced by AASHTO's Technical Committee on Roadside Safety (TCRS) that by the year 2019 all bridge rails installed on the national highway system must meet *MASH* TL3 conditions. It is uncertain how this will affect the eligibility of the modified Illinois Two-Tube bridge rail, since this system is only intended to be used on the local road system. Based on the results of FEA, the bridge rail system was also shown to meet crash performance and safety criteria set forth in *MASH* for TL3 conditions; however, the system should not be implemented as a *MASH* TL3 system without verifying its performance through full-scale crash testing. The system is not likely to meet *MASH* TL4 performance since it expected that the height of the barrier (i.e., 32 inches) is not sufficient to prevent the single unit truck from rolling over the bridge rail during impact.

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