

Preliminary Investigation

Caltrans Division of Research and Innovation

Compliance Crash Testing of Side Mounted Bridge Rail

Written by

Vue Her, Division of Research and Innovation

March 6, 2013

The Caltrans Division of Research and Innovation (DRI) receives and evaluates numerous research problem statements for funding every year. DRI conducts Preliminary Investigations on these problem statements to better scope and prioritize the proposed research in light of existing credible work on the topics nationally and internationally. Online and print sources for Preliminary Investigations include the National Cooperative Highway Research Program (NCHRP) and other Transportation Research Board (TRB) programs, the American Association of State Highway and Transportation Officials (AASHTO), the research and practices of other transportation agencies, and related academic and industry research. The views and conclusions in cited works, while generally peer reviewed or published by authoritative sources, may not be accepted without qualification by all experts in the field.

Summary

Background

The California Department of Transportation (Caltrans) is constantly faced with Right-of-Way issues and other limitations that make it impossible to place standard bridge rails mounted to the top of the bridge deck. The Caltrans Highway Safety Features New Products Committee, a committee comprised of representatives from all Divisions within Caltrans, recognizes that crash testing of a side mounted TL-4 rated rail has a high priority. Thus there is a need for a side-mounted bridge rail that can be used in areas where the posted speed limit will be more than 70 km/hr (45 mph). Crash testing will be performed according to current crash test guidelines, Manual for Assessing Safety Hardware 2009 (MASH), Test Level 4 (TL-4) for longitudinal barriers. Successful crash testing of the side mounted bridge rail will allow the Department to utilize the barrier on bridges with limitations, ensuring that it will safely handle impacting vehicles by satisfying all three areas of MASH 2009's evaluation criteria: structural adequacy, occupant risk, and vehicle trajectory.

The Department has several side mounted bridge rails in their inventory but none of the barriers have been crash tested under either the current MASH 2009 guidelines or previous NCHRP Report 350 guidelines. Because the existing side-mounted bridge rails do not meet current LRFD standards and specifications, using them potentially exposes the Department to tort liability. Thus there is an urgent need for a side mounted barrier to fill the vacuum created by the removal of non-crash test compliant side-mounted bridge rail barriers.

Using a bridge rail without verifying its performance through crash testing also poses a tort liability risk. The side-mounted bridge rail needs to be crash tested and evaluated to ensure that it meets the MASH 2009 criteria as required by the Federal Highway Administration (FHWA).

The objective of the research project is to come up with a side mounted bridge rail that will meet the evaluation criteria of MASH 2009 TL-4 for longitudinal barriers. Test level 4 consists of three crash tests as follows:

1. An 1,100 kg (2,420 lbs) car at 100 km/hr and a 25 degree impact angle.
2. A 2,270 kg (5,000 lbs) truck at 100 km/hr and a 25 degree impact angle.
3. A 10,000 kg (22,000 lbs) single-unit truck at 90 km/hr and a 15 degree impact angle.

A representative section of the bridge rail will be constructed at a Caltrans Dynamic Test Facility in West Sacramento. The Roadside Safety Research Group will conduct the three crash tests on the test article, evaluate the results, redesign and retest if necessary, and determine if the tests meet the criteria set forth in MASH 2009.

Summary of Findings

Several locations were searched for crash tests on side mounted bridge rails. No similar products were found that had been tested to MASH 2009 TL-4. There were two products that were tested to NCHRP Report 350 TL-4 guidelines and accepted by FHWA (See Attachments 1, 2, & 3). They were designed by the University of Nebraska-Lincoln, Midwest Roadside Safety Facility. Although these products were tested to NCHRP Report 350 guidelines, they were only designed for use on transverse, glue-laminated timber bridge decks. In an email to the Midwest Roadside Safety Facility, Dr. Ronald K. Faller responded that the "...two system(s) could be adapted over to concrete decks with consideration of edge deck design and adequate reinforcement." (See Attachment 4) Caltrans Structures Design will have to evaluate the two products from Midwest to determine if they can be used on California bridge decks. Even though there are two products accepted by FHWA, they were tested under the old guidelines. Caltrans Structures Design has requested a side mounted bridge rail that complies with TL-4 to assure that it will safely handle impacting vehicles by satisfying the requirements of MASH 2009.

The following locations were searched:

- *Federal Highway Administration Website*
- *National Administration Website*
- *Caltrans Research*
- *A search of TRIS RIP to find research in progress yielded no relevant results*
- *General Internet Search*

National Research

- Federal Highway Administration Website: A search for side mount bridge rails yielded a few products but none that were tested to MASH 2009 TL-4. The side mounted bridge rails that were most similar were:
 1. Illinois Type SM - Steel Bridge Rail Side Mounted (NCHRP Report 350 TL-4)
 2. Oregon 2 - Tube Side Mount (NCHRP Report 350 TL-4)
 3. Midwest Steel Thrie-Beam with Upper Channel TCB8000 (NCHRP Report 350)
 4. Midwest TL-4 Glulam Timber Bridge Railing (NCHRP Report 350)

- National Transportation Library: Multiple searches using keywords "bridge", "rail", "barrier", "side mount", "TL-4", "Test Level 4", "2009", and "MASH" in combination yielded no products that were tested to MASH 2009 TL-4.

Caltrans Research

- A search of the Caltrans Division of Research and Innovation's Research Report Website yielded no side mount bridge rails that were tested to MASH TL-4.
- Caltrans has a couple side mount bridge rails but none of the barriers have been crash tested under the recent MASH 09 or old NCHRP Report 350 guidelines.

The following tables lists known side mounted bridge rails at test levels 1 through 4.

TEST LEVEL 1

Name	Location	NCHRP Report 230	NCHRP Report 350	MASH 2009	Test Level	FHWA Acceptance	Contact
W-Beam Breakaway Timber Post Railing	N/A		x		1	NO	Ronald K. Faller, Midwest, (402)472-6864
W-Beam Breakaway Steel Post Railing	N/A		x		1	NO	Ronald K. Faller, Midwest, (402)472-6864
Glacier Removable Bridge Railing (SBD04a)	N/A		x		1	Yes, HSA-10/B64-D	Dan Van Gilder, FHWA's Eastern Federal Lands Division, (703)404-6361

TEST LEVEL 2

Name	Location	NCHRP Report 230	NCHRP Report 350	MASH 2009	Test Level	FHWA Acceptance	Contact
Box Beam Rail (W-Beam Backed with Steel Beam)	Ohio		x		2		Matt Shamis, FHWA-Ohio Division, (614)280-6847
Oregon Thrie-Beam Side Mount	Oregon		x		2		Antony P. Stratis, Tech Center Bridge Manager, (503)731-8490
California Thrie Beam Bridge Rail	California	x			2		Nahed Abdin, Caltrans, (916)227-8805
California Type 18	California		x		2		Nahed Abdin, Caltrans, (916)227-8805
California Side Mount Type 115 Rail	California	x			2		Nahed Abdin, Caltrans, (916)227-8805
California Type 116 Rail	California		x		2		Nahed Abdin, Caltrans, (916)227-8805
California Type 117 Rail	California		x		2		Nahed Abdin, Caltrans, (916)227-8805
Glu-Lam Rail with Steel Box Attachment, Side Mount	N/A		x		2	Yes, HSA-10/B-138	Ronald K. Faller, Midwest, (402)472-6864

TEST LEVEL 3

Name	Location	NCHRP Report 230	NCHRP Report 350	MASH 2009	Test Level	FHWA Acceptance	Contact
Side Mount W Beam	Michigan		x		3	NO	Steve Beck, Michigan DOT, (517)373-0097
Timber Rail 3 Bridge Rail	Oklahoma		x		3	N/A	N/A
MGS Bridge Rail (SBO02c)	N/A			x	3	NO	Ronald K. Faller, Midwest, (402)472-6864

TEST LEVEL 4

Name	Location	NCHRP Report 230	NCHRP Report 350	MASH 2009	Test Level	FHWA Acceptance	Contact
Illinois Type SM - Steel Bridge Rail Side Mounted	Illinois		x		4	NO	Thomas J. Domagalski, Illinois DOT, (217)782-2125
Oregon 2 - Tube Side Mount (OR Two Tube)	Oregon		x		4	NO	Antony P. Stratis, Tech Center Bridge Manager, (503)731-8490
Steel Thrie-Beam Rail with Upper Channel, TCB8000 Design	N/A		x		4	Yes, HSA-10/B-138	Ronald K. Faller, Midwest, (402)472-6864
TL-4 Glulam Timber Bridge Railing (SBD01d)	N/A		x		4	Yes, HSA-10/B-138	Ronald K. Faller, Midwest, (402)472-6864

General Internet Search

Links:

Unofficial table of bridge rails:

- <http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=browse&all=1>

Side mount bridge rails listed in the above table without acceptance letters:

1. <http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=view&rating=61>
2. <http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=view&rating=51>
3. <http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=view&rating=121>
4. <http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=view&rating=122>
5. <http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=view&rating=36>
6. <http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=view&rating=38>
7. <http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=view&rating=117>
8. <http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=view&rating=120>

Side mount bridge rails listed in the above table with acceptance letters (under NCHRP Report 350 guidelines):

1. <http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=view&rating=29>
2. <http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=view&rating=30>
3. <http://guides.roadsafellc.com/bridgeRailGuide/index.php?action=view&rating=113>

Federal Highway Administration Website (FHWA)

- <http://www.fhwa.dot.gov/bridge/bridgerail/index.cfm>
- http://search.fhwa.dot.gov/search?q=side+mounted+bridge+rail&btnG.x=0&btnG.y=0&ie=&site=fhwa_web&output=xml_no_dtd&client=fhwa_web&lr=&proxystylesheet=fhwa_web&oe
- http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/bridgerailings/

General Internet Searches

- <http://www.google.com/search?q=bridge+rail+barrier&ie=utf-8&oe=utf-8&aq=t&rls=org.mozilla:en-US:official&client=firefox-a&safe=active>

- <http://rip.trb.org/search/search.aspx?f1=k%3A%3AKeywords+%28Title%2C+Abstract%2C+or+Index+Terms%29&sc=xx%3A%3AAll+Categories&t1=barrier+bridge+rail>
- <http://ntlsearch.bts.gov/repository/search.do?new=&d=dc+OR+po&b1=1&t1=barrier&f1=kw&b2=1&t2=bridge&f2=kw&b3=1&t3=rail&f3=kw&Submit=Search&f10=rt&b10=1&t10=&f12=yr&b12=1&t12=&f13=mt&b13=1&t13=&f14=ln&b14=1&t14=&f15=gc&b15=1&t15=&s=yr&o=1&z=50>

Attachment 1



U.S. Department
of Transportation
**Federal Highway
Administration**

400 Seventh St., S.W.
Washington, D.C. 20590

August 4, 2005

In Reply Refer To: HSA-10/B-138

Ronald K. Faller, Ph.D., P.E.
Research Assistant Professor
Midwest Roadside Safety Facility
University of Nebraska-Lincoln
527 Nebraska Hall
Lincoln, Nebraska 68588-0529

Dear Dr. Faller:

On August 19, 2004, you sent Mr. A. George Ostensen, former Associate Administrator of Federal Highway Administration's (FHWA) Office of Safety, copies of two Midwest Roadside Safety Facility (MwRSF) test reports which detailed the design and testing of four bridge railings and their respective transitions for use on transverse, glue-laminated timber bridge decks. You included videotapes and digitized videos of the tests you conducted and requested FHWA acceptance of the designs for use on the National Highway System.

Staff members have belatedly reviewed the information you submitted and agree that the bridge rail designs shown as Enclosures 1 and 2 meet all evaluation criteria for a test level 2 (TL-2) bridge railing and those shown as Enclosures 3 and 4 meet TL-4 criteria. I understand that detailed test results and design drawings for these railings and their respective transitions can be obtained directly from you through the MwRSF.

Although the cargo bed separated from the single-unit truck frame in the TL-4 transition test with the wood post system, film analysis revealed that this separation was the result of inadequate connections between the cargo box and the truck frame and was not attributable to any contact with the transition elements. In the test on the bridge railing itself, the single-unit truck was contained and redirected with no separation of the cargo box from its frame. Nonetheless, designers should be cautioned against using a relatively low bridge railing at locations where failure to retain a cargo box could have severe consequences.

Sincerely yours,

/original signed by/

John R. Baxter, P.E.
Director, Office of Safety Design
Office of Safety

4 Enclosures



Attachment 1

135

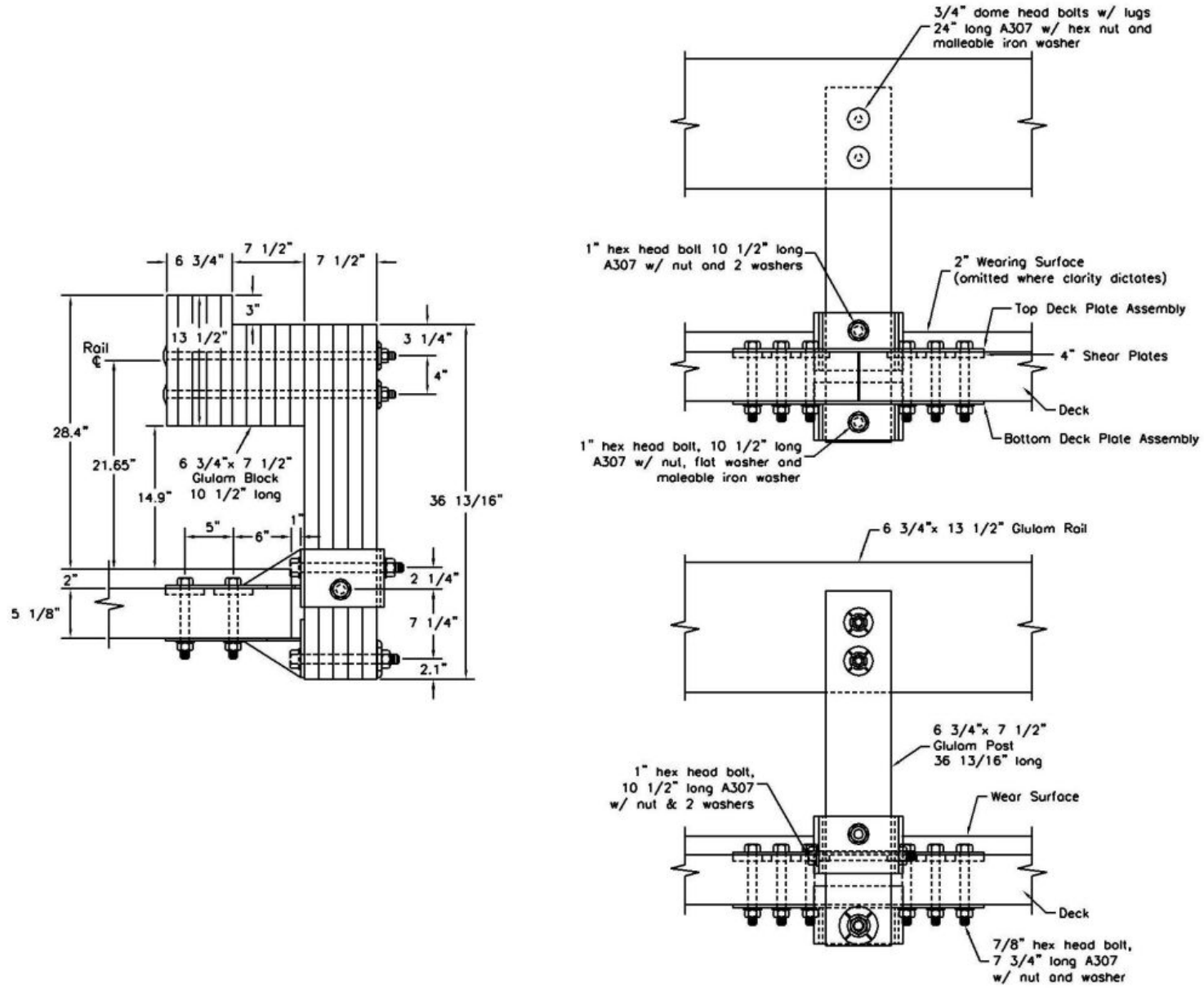


Figure 91. Bridge Railing Design Details - Wood System

Attachment 1

56

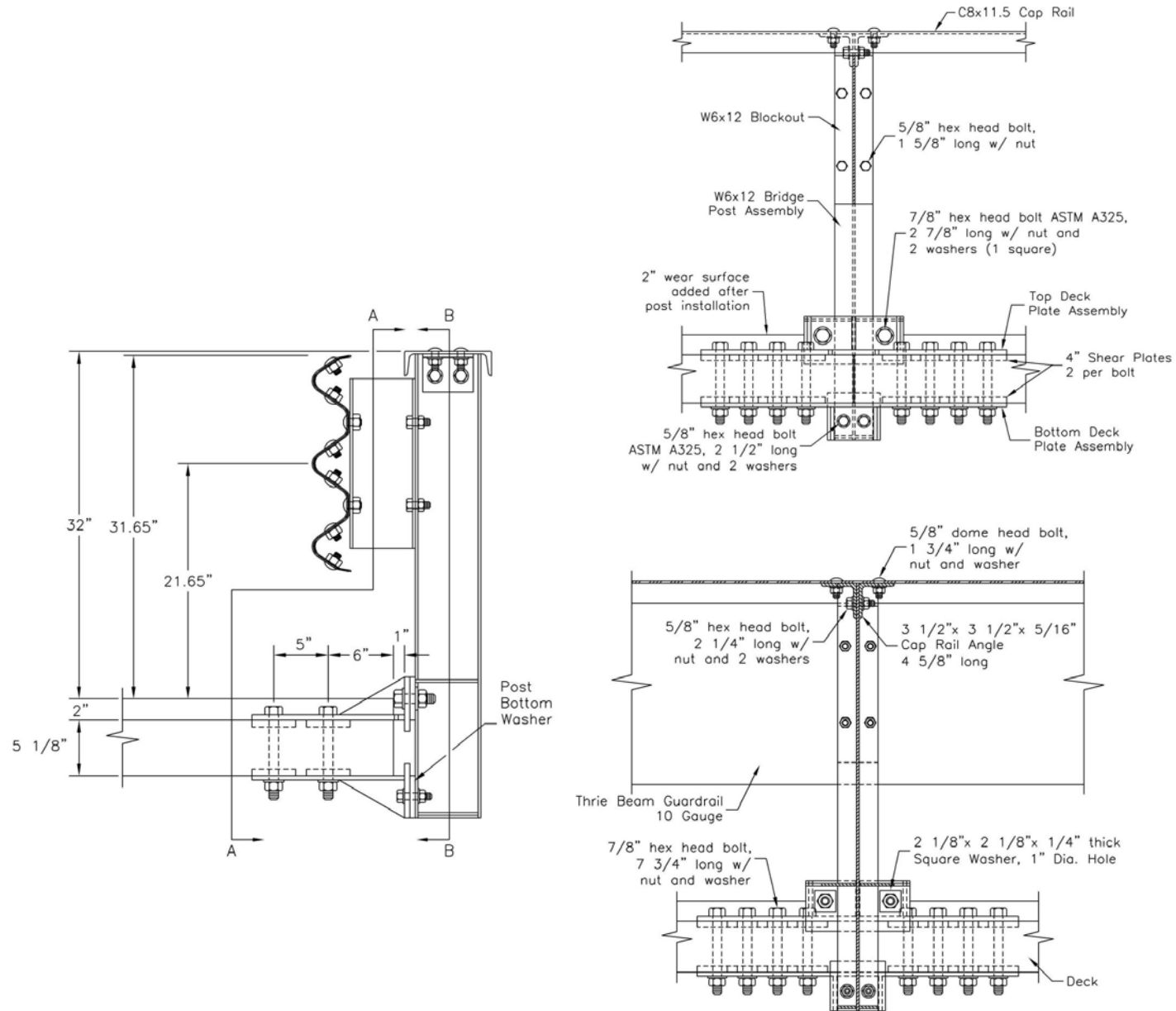


Figure 33. Bridge Railing Design Details - Steel System

Attachment 1

83

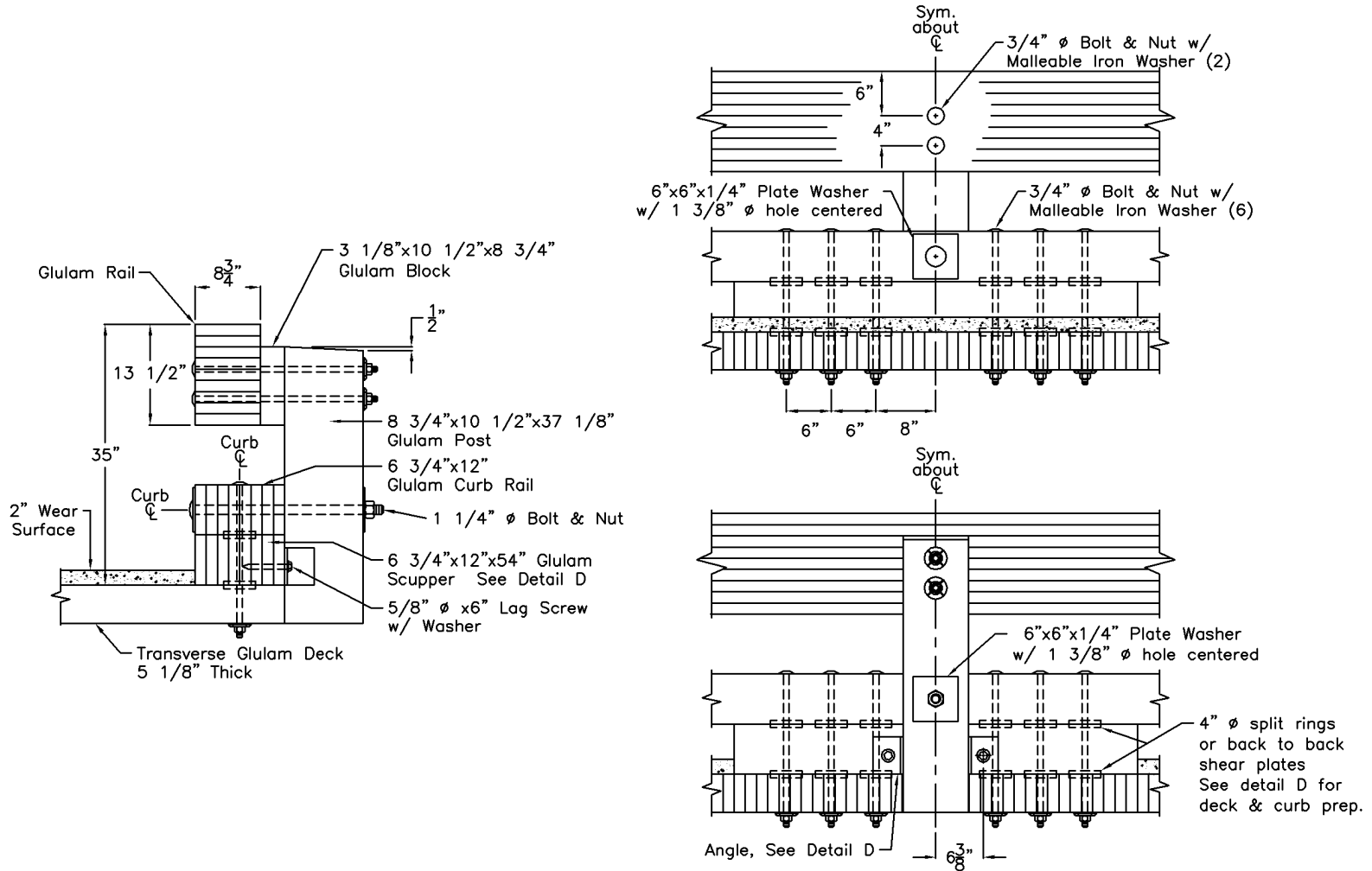


Figure 52. Bridge Railing Design Details - Wood System

Attachment 1

188

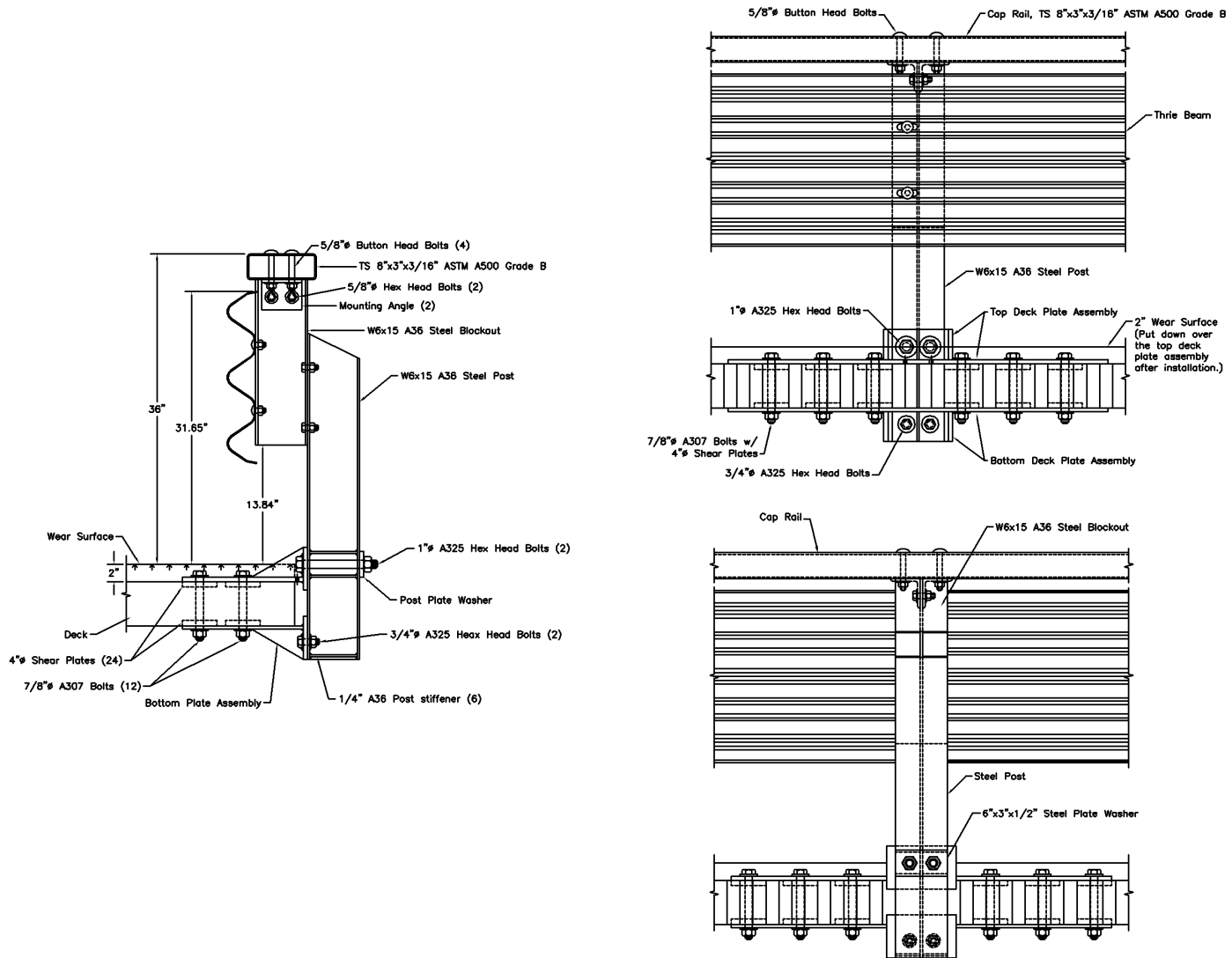


Figure 121. Bridge Railing Design Details - Steel System

Performance Level 2 and Test Level 4 Bridge Railings for Timber Decks

BARRY T. ROSSON, RONALD K. FALLER, AND MICHAEL A. RITTER

The Midwest Roadside Safety Facility, in cooperation with the Federal Highway Administration and U.S. Department of Agriculture Forest Service, Forest Products Laboratory, developed and tested two bridge railings for use on longitudinal timber bridge decks: (a) a steel railing system (TBC-8000) and (b) a glulam timber railing system (GC-8000). The test for the TBC-8000 was conducted according to Performance Level 2 as specified in the AASHTO *Guide Specifications for Bridge Railings* (1989). The tests for the GC-8000 were conducted according to Test Level 4 as specified in *NCHRP Report 350*. The safety performance of each of the bridge railings was acceptable according to each applicable crash test criterion. Both railings provide aesthetically pleasing and economical alternatives for use on higher-service-level timber bridges.

Most crashworthy bridge railing systems have been developed using materials such as concrete, steel, and aluminum. In addition, most of these railing systems have been constructed on reinforced concrete decks. However, many of the existing bridge railings have not been adapted for use on timber decks. The demand for crashworthy railing systems on timber decks has become increasingly important with the increased use of timber bridges on local roads and secondary highways.

Only recently have researchers begun to develop crashworthy railing systems for timber bridge decks. Further, all of these railing systems were designed for low-to-medium service-level bridges. For timber to be a viable material in the new construction of higher service-level bridges, additional bridge railing systems must be developed and crash-tested for timber bridges.

LITERATURE REVIEW

In 1988, the Texas Transportation Institute (TTI) conducted a safety performance evaluation of the Missouri thrie-beam bridge rail system and transition for the Missouri Highway and Transportation Department (1). The bridge rail consisted of W6 × 20 steel posts spaced on 1.90 m (6 ft 3 in.) centers and mounted to the surface of a reinforced concrete bridge deck. A 10-gauge thrie-beam rail was mounted to the traffic-side face of the posts without spacer blocks. To further strengthen the rail, a C8 × 11.5 structural steel channel was mounted to the top of the steel posts at a height of 77.8 cm (2 ft 6 7/8 in.). Two full-scale crash tests were conducted on the bridge rail according to *NCHRP Report 230* (2). The first test was performed with a 823-kg (1,815-lb) minicompact with impact conditions of 95.9 km/hr (59.6 mph) and 15.0 degrees. The second test was performed with a 2,039-kg (4,495-lb) sedan with impact con-

ditions of 98.0 km/hr (60.9 mph) and 24.0 degrees. According to TTI researchers, the Missouri thrie-beam bridge rail was acceptable according to *NCHRP Report 230* criteria (2).

In 1988, Southwest Research Institute (SwRI) performed an evaluation of a longitudinal glulam timber and sawed lumber curb railing system attached to a longitudinal spike-laminated timber deck (3). The system evaluated at SwRI was constructed and tested with sawed lumber post 20.3 cm (8 in.) wide × 30.5 cm (12 in.) deep. The system also had been constructed with a nonstandard-size @s-lam rail 15.2 cm (6 in.) × 27.3 cm (10 3/4 in.). The curb rail had dimensions of 15.2 cm (6 in.) × 30.5 cm (12 in.) and was attached to the deck with four 1.9-cm (3/4 in.) -diameter ASTM A325 bolts. Two crash tests were conducted according to the AASHTO *Guide Specifications for Bridge Railings* (4): the first was a PL1 test using a 2,383-kg (5,254-lb) pickup traveling at a speed of 76.4 km/hr (47.5 mph) and at an angle of 20 degrees; the second was a PL2 test using an 825-kg (1,818-lb) minicompact traveling at a speed of 95.3 km/hr (59.2 mph) and at an angle of 20 degrees. Although the system met AASHTO PL1 requirements, delamination of several of the deck timbers and minor pull-out of several spikes was observed. Although this system was widely used, and was the only available crash-tested railing for timber bridges, the demand continued for crashworthy bridge railings that would not damage the timber decks and that would be adaptable for use on other timber decks.

In the early 1990s, Forest Product Laboratory and Midwest Roadside Safety Facility (MwRSF) researchers developed and tested three PL1 bridge railings (two glulam timber railing systems and one steel railing system) for use on longitudinal timber decks (5,6). This research effort provided several aesthetically pleasing and economical bridge railings for timber bridge decks on low-to-medium service-level highways. The geometry of the PL1 thrie-beam "steel system" railing was essentially unchanged from the previously tested California thrie-beam bridge rail (7). Therefore, it was considered unnecessary to perform a test with the minicompact sedan (which was successfully tested during the California development) because there was no potential for wheel snagging or concern for occupant risk. Because the basic geometry of the PL1 glulam timber "curb system" railing was unchanged from the timber system tested by SwRI (3), it was deemed unnecessary to perform the test with a minicompact sedan as well. However, the structural components and load transfer mechanisms for both railings were significantly modified, thus requiring crash testing with a 2,449-kg (5,400-lb) pickup truck.

RESEARCH OBJECTIVES

Following the successful development of the three MwRSF PL-1 bridge railings on longitudinal timber decks, a research project was planned to further develop aesthetic and economical bridge railings

B. T. Rosson and R. K. Faller, Midwest Roadside Safety Facility, Civil Engineering Department, 1901 Y Street, Building C, University of Nebraska-Lincoln, Lincoln, Nebr. 68588-0601. M. A. Ritter, U.S. Department of Agriculture Forest Service, Forest Products Laboratory, One Gifford Pinchot Drive, Madison, Wis, 53705.

for timber bridges on higher service-level roadways. The Midwest Roadside Safety Facility in cooperation with the USDA Forest Service, Forest Products Laboratory, and the Federal Highway Administration, developed a PL2 (2) thrie-beam railing and a TL-4 (8) glulam timber railing that would be compatible with the existing types of longitudinal timber bridge decks. The first bridge railing was a steel system constructed using thrie-beam with a channel attached above spacer blocks (TBC-8000). The second railing was constructed using a glulam timber rail with a curb mounted on scupper blocks (GC-8000).

EVALUATION CRITERIA

Throughout the development of the TBC-8000, crash test criteria of the 1989 AASHTO *Guide Specifications for Bridge Railings (4)* were used. To be considered an AASHTO PL2 bridge railing, the railing must satisfy the safety requirements from three full-scale vehicle crash tests. The required PL2 tests are:

1. An 816-kg (1,800-lb) minicompact traveling at 96.6 km/hr (60 mph) and 20 degrees;
2. A 2,449-kg (5,400-lb) pickup traveling at 96.6 km/hr (60 mph) and 20 degrees; and
3. An 8,165-kg (18,000-lb) single-unit truck traveling at 80.5 km/hr (50 mph) and 15 degrees. The guide specifications require that the full-scale crash tests be conducted and reported in accordance with *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances (2)*.

NCHRP Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features (8) was published and adopted by the FHWA while the GC-8000 was being developed. Consequently, the GC-8000 railing was evaluated using the TL4 crash test criteria. The required TL4 tests are:

1. An 820-kg (1,808-lb) minicompact traveling at 100 km/hr (62.1 mph) and 20 degrees;
2. A 2,000-kg (4,409-lb) pickup traveling at 100 km/hr (62.1 mph) and 25 degrees; and
3. An 8,000-kg (17,637-lb) single-unit truck traveling at 80 km/hr (49.7 mph) and 15 degrees.

TBC-8000 SYSTEMS

System Development

The previously accepted AASHTO PL1 "steel system" for timber decks (5.6) was selected as the basis for the design of the AASHTO PL2 steel bridge railing. Because the Missouri combination steel railing system successfully met the *NCHRP Report 230* safety performance evaluation, and would likely meet the AASHTO PL2 pickup truck crash test criteria as well, concepts from the Missouri railing were used in the design of the new PL2 railing for timber bridge decks.

The minicompact vehicle test conducted on the Missouri thrie-beam bridge railing was performed at 15 degrees, as the *NCHRP Report 230* evaluation criteria require (2). Thus, the test results would have been similar if the Missouri railing system had been conducted at 20 degrees, because there was no observable tendency for the vehicle to snag or override the bridge railing. Also, because the Missouri bridge railing successfully met the *NCHRP Report 230*

strength test using a 2,039-kg (4,495-lb) sedan at 98.0 km/hr (60.9 mph) and 24.0 degrees, the AASHTO PL2 strength test (with a 2,449-kg (5,400-lb) pickup traveling at 96.6 km/hr (60 mph) and 20 degrees) would have yielded similar results to the sedan strength test because the impact severity of the sedan crash test was determined to be 132 kJ (97 k-ft), whereas the impact severity for the pickup test was only 103 kJ (76 k-ft). Although the center of mass of the pickup is higher than that of the sedan and would produce slightly higher bending moments in the posts if the impact severities were the same, the actual lower impact force of the pickup test, even when applied at a slightly higher level, would not produce moments of sufficient magnitude to overcome the difference in severity levels. Therefore, with the TBC-8000 consisting of similar structural members as the Missouri railing, only the 8,165-kg (18,000-lb) single-unit truck crash test would have to be conducted for the new railing to meet PL2 crash test criteria.

It was concluded that the PL1 steel system design should be stiffened to meet AASHTO PL2 standards since three of the posts had significant deformation from the PL1 pickup test (5,6). In addition, the Missouri thrie-beam railing had 15.9 cm (6.25 in.) of permanent set deflection when hit by the sedan (1). Therefore, a C8 × 11.5 steel channel was mounted above the spacer block of the PL1 steel system (Figure 1) to strengthen the bridge rail and meet PL2 strength standards. The top of the steel channel section has a mounting height of 84.5 cm (2 ft 9¼ in.) to provide clearance above the thrie-beam. This provides vertical support for the bottom of the truck box during impact, thus reducing the amount of roll motion of the truck box.

Design Details

The TBC-8000 bridge railing consists of four major components: (a) structural steel posts and spacer blocks; (b) steel thrie-beam rail; (c) structural steel channel rail; and (d) structural steel mounting plates. An illustration of the TBC-8000 bridge railing is shown in Figure 1.

Fifteen galvanized ASTM A36 W6 × 15 structural steel posts 93.3 cm (3 ft 3¼ in.) long were used to support the steel railing. The steel posts were attached to the longitudinal glulam timber deck with ASTM A36 structural steel mounting plates. Fifteen steel mounting plates 1.9 cm (3/4 in.) thick, 27.3 cm (10¼ in.) deep, and 61.0 cm (24 in.) long were attached to the deck with two ASTM A722 high-strength bars 2.5 cm (1 in.) in diameter and 1.37 m (4 ft 6 in.) long, spaced at 40.6 cm (16 in.) and located 7.6 cm (3 in.) below the top surface of the deck. Design details for the bearing plates located at the other end of the rods are included in a study by Ritter et al. (6). Each steel post was bolted to a steel mounting plate with four 2.2 cm (7/8 in.) diameter ASTM A325 galvanized hex head bolts. Four recessed holes were cut into the edge of the timber deck so the steel mounting plates would bolt flush against the vertical deck surface. The lower rail consisted of a 10-gauge thrie-beam mounted 78.4 cm (2 ft 6⅞ in.) above the timber deck surface. The thrie-beam rail was offset 15.2 cm (6 in.) away from the posts with galvanized ASTM A36 W6 × 15 structural steel spacer blocks 58.7 cm (1 ft 11 ⅞ in.) long. The upper rail consisted of galvanized ASTM A36 C8 × 11.5 structural steel channel sections attached to the top of the steel spacer blocks. The top of the channel rail was 84.5 cm (2 ft 9¼ in.) above the asphalt surface. The channel rail sections were attached to the spacer blocks with 3½ × 3½ × ⅞ ASTM A36 structural steel angles. Each channel rail section was spliced together with ASTM A36 structural steel splice plates.

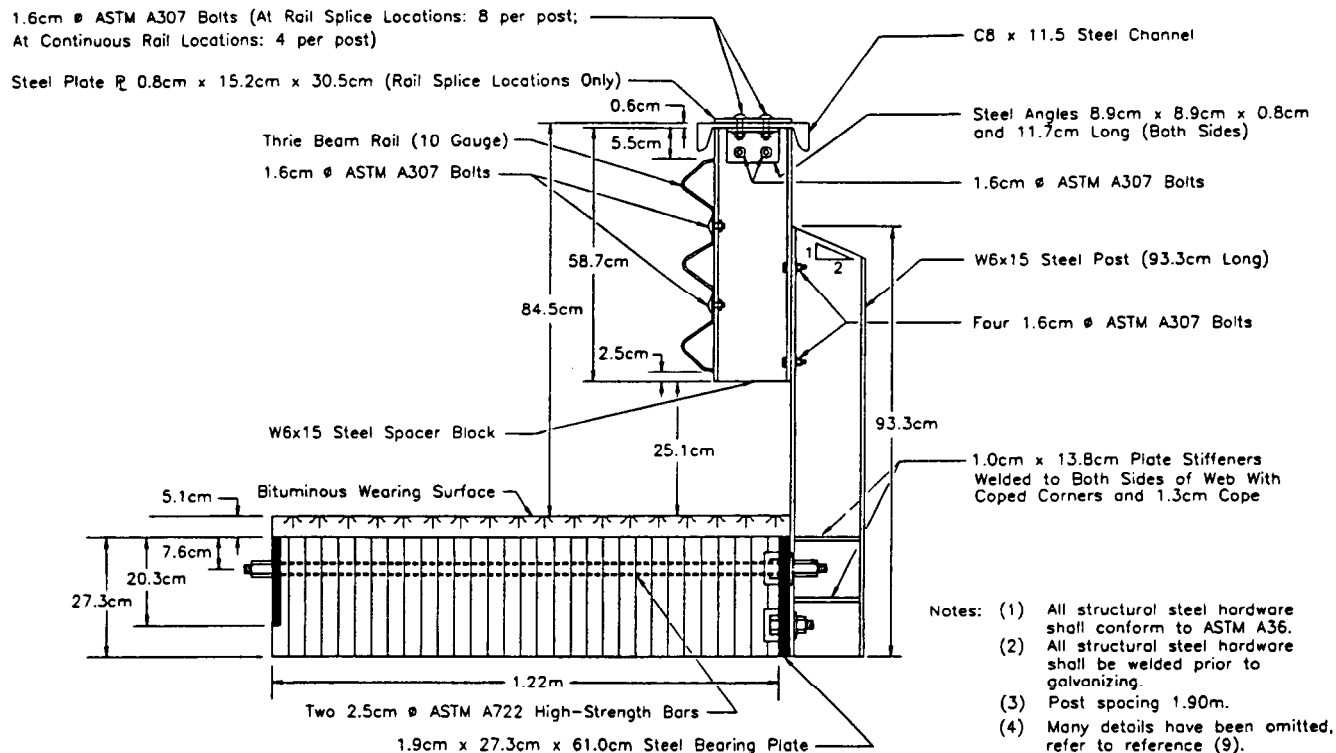


FIGURE 1 Thrie-beam with channel bridge railing (TBC-8000).

An approach guardrail transition was constructed on the upstream end of the TBC-8000 bridge railing. Details of the approach guardrail transition can be found in the Forest Product Laboratory Report on the TBC-8000 (9).

The rail was attached to a longitudinal glulam timber deck supported by concrete abutments. A full-size simulated timber bridge system was constructed at the Midwest Roadside Safety Facility to simulate an actual timber bridge installation. The inner three concrete bridge supports had center-to-center spacings of 5.71 m (18 ft 9 in.), and the outer two spacings were 5.56 m (18 ft 3 in.). The longitudinal glulam timber deck consisted of 10 rectangular panels measuring 1.22 m (3 ft 11 $\frac{1}{4}$ in.) wide, 5.70 m (18 ft 8 $\frac{1}{2}$ in.) long, and 27.3 cm (10 $\frac{3}{4}$ in.) thick. It was constructed so that two panels formed the width and five panels formed the length of the installation. The longitudinal glulam timber deck was fabricated with Combination No. 2 West Coast Douglas Fir and treated with pentachlorophenol in heavy oil to a minimum net retention of 9.61 kg/m³ (0.6 lb/ft³) as specified in American Wood-Preservers' Association Standard C14 (10). At each longitudinal midspan location of the panels, stiffener beams were bolted transversely across the bottom of the deck per AASHTO bridge design requirements. The stiffener beams measured 13.0 cm (5 $\frac{1}{8}$ in.) wide, 15.2 cm (6 in.) thick, and 2.44 m (8 ft) long. The timber deck had a 5.1-cm (2-in.) asphalt surface on top to represent actual field conditions.

Computer Simulation

After the preliminary design of the TBC-8000, computer simulation modeling with *BARRIER VII* (11) was performed to analyze the dynamic performance of the bridge railing before full-scale crash test-

ing. The simulation was conducted modeling a 8,165-kg (18,000-lb) single-unit truck striking the rail at 80.5 km/hr (50 mph) and 15 degrees.

The simulation results indicated that the TBC-8000 bridge railing would successfully redirect the 8,165-kg (18,000-lb) single-unit truck. In addition, the modeling indicated that all structural hardware would remain functional during the impact. The maximum dynamic deflections of the C-rail and thrie-beam were 34.8 cm (13.7 in.) and 29.2 cm (11.5 in.), respectively. The maximum permanent set deflections of the C-rail and thrie-beam were 17.8 cm (7.0 in.) and 15.2 cm (6.0 in.), respectively. The maximum 0.010-sec average lateral and longitudinal decelerations were 2.7 and 2.0g, respectively. The peak 0.050-sec average impact force perpendicular to the bridge railing was approximately 222 kN (50 kips). The truck became parallel to the bridge railing at 0.350 sec. At 0.680 sec, the truck exited the bridge railing at an angle of 11.4 degrees.

Full-Scale Crash Test

Test FSTC- 1 [8, 165-kg (18,000-lb), 76.3 km/hr (47.4 mph), 16.1 degrees] struck the bridge railing at Post No. 4 (Figure 2). A summary of the test results and the sequential photographs is presented in Figure 3.

After the initial impact with the bridge railing, the right-front corner of the bumper and quarter panel crushed inward. The truck became parallel with the rail at 0.399 sec with a velocity of 66.6 km/hr (41.4 mph). At 0.523 sec, the front-end of the truck began to yaw away from the rail, and at 0.622 sec, the truck box reached a maximum clockwise roll angle of approximately 18 degrees. The truck exited the bridge rail at approximately 1.504 sec and 1.8 degrees. The effective coefficient of friction was determined to be 0.31.

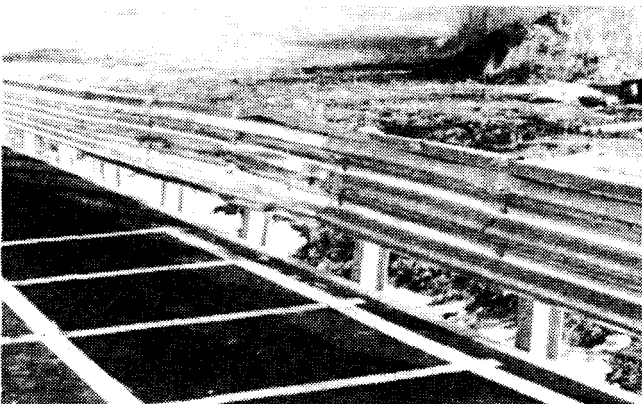
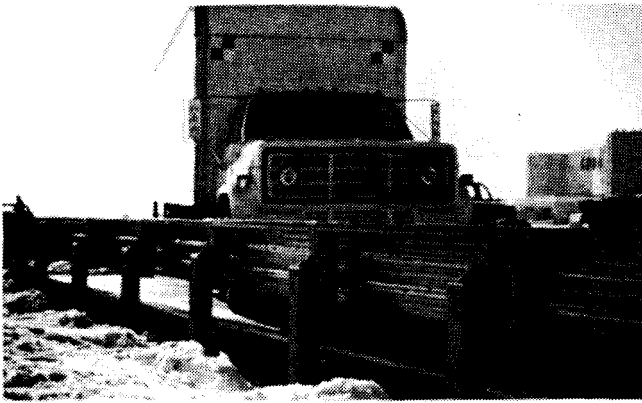


FIGURE 2 Impact location, vehicle damage, and bridge rail damage, Test FSTC-1.

Vehicle damage was relatively minor and was limited to the right-front corner of the truck cab, box, and front bumper (Figure 2). The bridge rail damage was moderate, consisting mostly of deformed thrie-beam sections, C-rail sections, and steel posts (Figure 2). Examination of the top and bottom surfaces of the timber deck laminations revealed no physical damage or separation.

The length of vehicle contact along the top of the C-rail was approximately 11.4 m (37 ft 6 in.). Physical evidence revealed that lateral buckling of the C-rail occurred between Post Nos. 4 and 5 (Figure 2). The physical damage to the thrie-beam rail revealed that approximately 7.6 m (25 ft) of rail was damaged. The maximum permanent set deflections of the C-rail and thrie-beam rail were 19.3 cm (7.6 in.) and 20.8 cm (8.2 in.), respectively.

Test FSTC-1 was evaluated according to the AASHTO PL2 criteria. The TBC-8000 bridge rail contained and smoothly redirected

the test vehicle with controlled lateral deflection of the bridge rail. There were no detached elements or fragments that showed potential for penetrating the occupant compartment or that presented undue hazard to other traffic. The test vehicle did not penetrate or ride over the bridge rail, and it remained upright during and after the crash. The occupant compartment was not damaged. The effective coefficient of friction, $\mu = 0.31$, was fair ($0.26 \leq \mu \leq 0.35$). The occupant risk values for occupant impact velocities and ridedown decelerations were satisfactory. The vehicle's trajectory revealed minimum intrusion into adjacent traffic lanes. The vehicle's exit angle from the bridge railing was less than 12 degrees.

GC-8000 SYSTEM

System Development

After the successful development and full-scale vehicle crash testing of the AASHTO PL1 curb system (5,6), it was determined that the PL1 bridge railing had adequate structural capacity and could be modified to meet a higher performance level. Therefore the AASHTO PL1 "curb system" was used as the basis for the design of the *NCHRP Report 350* TL4 glulam railing.

The glulam rail previously tested at SwRI (3) was crash-tested using an 825-kg (1,818-lb) minicompact at 95.3 km/hr (59.2 mph) and 20 degrees and a 2,383-kg (5,254-lb) pickup at 76.4 km/hr (47.5 mph) and 20 degrees. Because the basic geometry of the PL1 curb system and the newly developed GC-8000 were essentially the same as the system tested at SwRI, repeating the minicompact sedan test was deemed unnecessary. However, to meet TL4 criteria, the 2,000-kg (4,409-lb) unballasted pickup test at 100 km/hr (62.1 mph) and 25 degrees and the 8,000-kg (17,637-lb) single-unit truck test at 80 km/hr (49.7 mph) and 15 degrees would have to be conducted.

Development of the GC-8000 consisted of re-sizing the structural components previously used with the AASHTO PL-1 curb system to withstand the higher impact forces generated from the TL4 crash test conditions. The components changed included the timber glulam rail, lumber posts, spacer and scupper blocks, and structural steel hardware. The PL1 curb system was constructed with sawed lumber Douglas Fir posts 20.3 cm (8 in.) wide and 20.3 cm (8 in.) deep, and the glulam rail was 17.1 cm (6¾ in.) wide and 26.7 cm (10½ in.) deep. However, computer simulation modeling indicated that the GC-8000 bridge rail posts needed to be 20.3 cm (8 in.) wide and 25.4 cm (10 in.) deep, and the glulam rail needed to be 17.1 cm (6¾ in.) wide and 34.3 cm (13½ in.) deep. The scupper blocks, used to support the sawed lumber curb rail and transfer the impact forces into the timber deck, were increased in length from 0.91 to 1.22 m (3 to 4 ft) and in depth from 14.0 to 19.1 cm (5½ to 7½ in.). The increase in length of the scupper blocks was required to accommodate the six ASTM A307 1.9 cm (¾ in.) diameter bolts needed to carry the increased impact forces into the deck. The increase in depth of the scupper blocks was used to accommodate a 5.1-cm (2-in.) asphalt-wearing surface placed on the timber deck.

Design Details

The GC-8000 consisted of five major components: (a) sawed lumber scupper blocks; (b) sawed lumber curb rail; (c) sawed lumber posts; (d) longitudinal glulam timber rail; and (e) timber spacer blocks. An illustration of the GC-8000 bridge railing is shown in Figure 4.

One timber scupper block was bolted to the timber deck at each post location with six ASTM A307 1.9 cm (¾ in.) diameter, 66.0

Attachment 2

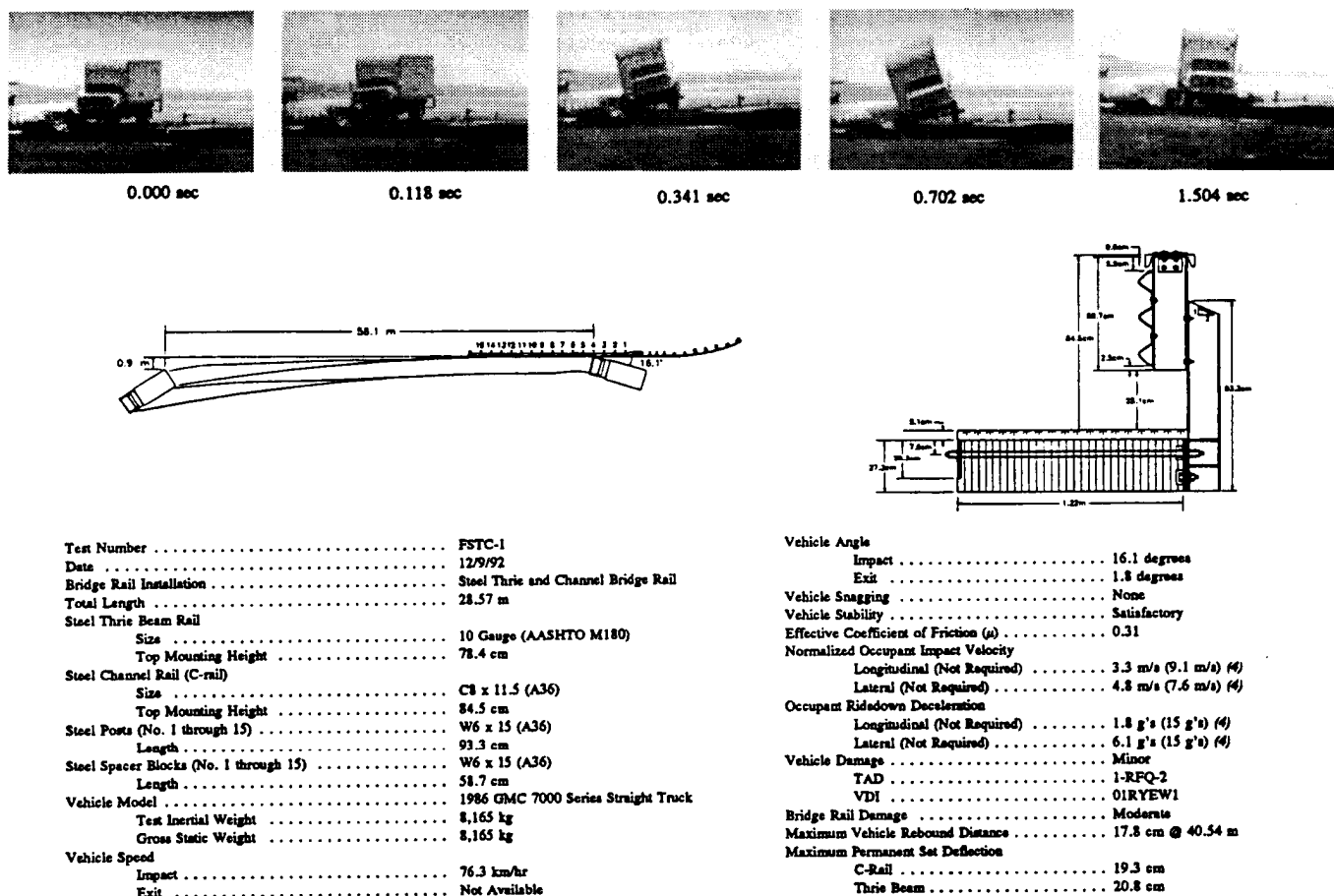


FIGURE 3 Summary of test results and sequential photographs, Test FSTC-1.

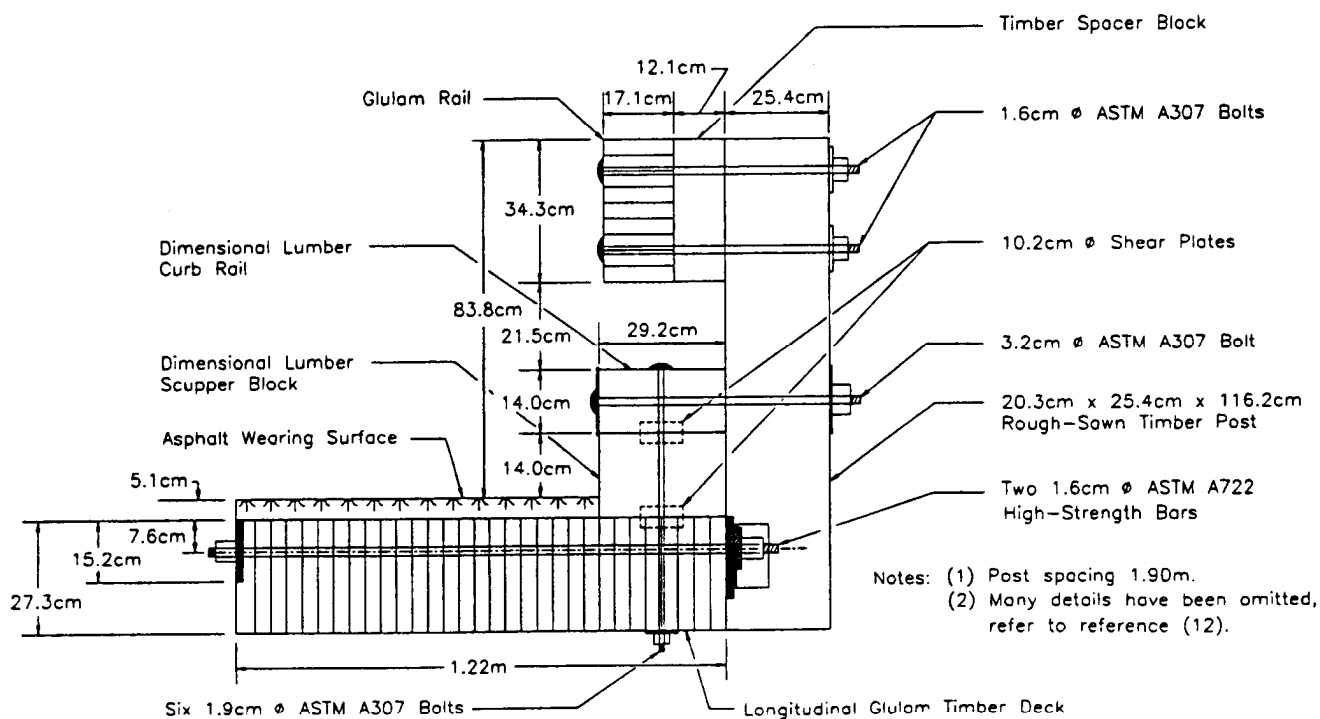


FIGURE 4 Glulam timber with curb bridge railing (GC-8000).

Attachment 2

cm (26 in.) long galvanized dome head bolts. The scupper blocks were fabricated with S4S No. 1 Grade Douglas Fir 19.0 cm (7½ in.) thick, 29.2 cm (11½ in.) wide, and 1.22 m (4 ft) long. They were attached to the curb rail and timber deck surface with 10.2 cm (4 in.) diameter shear plate connectors. The curb rail was fabricated with S4S No. 1 Grade Douglas Fir 14.0 cm (5½ in.) deep and 29.2 cm (11½ in.) wide, with the top of the curb rail positioned 28.0 cm (11 in.) above the asphalt-wearing surface. One ASTM A307 3.2 cm (1¼ in.) diameter, 63.5 cm (25 in.) long dome head bolt was used to attach each of the 15 bridge posts to the curb rail. Two 1.37 m (4 ft 6 in.) long high-strength bars were placed 55.9 cm (22 in.) apart transversely through the outer timber deck panel at each post. Fifteen No. 1 Grade rough-sawn lumber Douglas Fir posts approximately 20.3 cm (8 in.) wide, 25.4 cm (10 in.) deep, and 1.16 m (3 ft 9¼ in.) long were used to support the upper glulam railing at a spacing of 1.90 m (6 ft 3 in.) on centers. The posts were treated to meet AWP Standard C 14 with 192.22 kg/m³ (12 lb/ft³) creosote (10). The longitudinal glulam rail was fabricated from Combination No. 2 West Coast Douglas Fir and treated in the same manner as the timber deck. The glulam rail was 17.1 cm (6¾ in.) wide and 34.3 cm (13½ in.) deep. The top mounting height of the glulam rail was 83.8 cm (2 ft 9 in.) above the asphalt-wearing surface. The glulam rail was offset from the posts with timber spacer blocks 12.1 cm (4¾ in.) thick, 20.3 cm (8 in.) wide, and 34.3 cm (13½ in.) deep. Two ASTM A307 1.6 cm (5/8 in.) diameter 61.0 cm (24 in.) long galvanized dome head bolts were used to attach the glulam rail to the timber posts. The rail was attached to a longitudinal glulam timber deck similar to the one used in the TBC-8000 crash test.

An approach guardrail transition was constructed on the upstream end of the GC-8000 bridge railing and crash-tested with a 2,041-kg (4,500-lb) sedan at 100.4 km/hr (62.4 mph) and 24.8 degrees. The crash test was evaluated according to the safety performance criteria provided in *NCHRP Report 230 (2)* and was acceptable. The sedan crash test was performed on the guardrail transition according to *NCHRP Report 230* criteria because at the time the transition was tested the GC-8000 was not intended to meet *NCHRP Report 350 (8)* TL4 criteria. Further details concerning the approach guardrail transition can be found in the Forest Product Laboratory Report on the GC-8000 (12).

Computer Simulation

After the preliminary design of the GC-8000, computer simulation modeling with BARRIER VII (11) was performed to analyze the dynamic performance of the bridge railing before full-scale crash testing. Computer simulations were conducted with an 8,165-kg (18,000-lb) single-unit truck hitting the rail at a speed of 80.5 km/hr (50 mph) and impact angle of 15 degrees, and with a 1,996-kg (4,000-lb) pickup truck traveling at a speed of 100 km/hr (62.1 mph) and having impact angle of 25 degrees.

The simulation results indicated that the GC-8000 bridge railing would satisfactorily redirect the 8,000-kg (17,637-lb) single-unit truck. In addition, all structural hardware would remain functional during the impact: the maximum dynamic and permanent set deflections of the glulam rail were 15.2 cm (6.0 in.) and 4.1 cm (1.6 in.), respectively. The maximum 0.010-sec average lateral and longitudinal decelerations were 3.3 and 2.1 g, respectively. The peak 0.050-sec average impact force perpendicular to the bridge railing was approximately 285 kN (64 kips). The truck became parallel to the bridge railing at 0.323 sec. At 0.625 sec, the truck exited the bridge railing at an angle of 12.3 degrees.

The simulation results also indicated that the railing would satisfactorily redirect the 2,000-kg (4,409-lb) pickup truck. In addition, all structural hardware would remain functional during the impact; the maximum permanent set and dynamic deflection of the glulam rail were 7.4 cm (2.9 in.) and 17.8 cm (7.0 in.), respectively. The maximum 0.010-sec average lateral and longitudinal decelerations were 13.2 and 10.9g, respectively. The peak 0.050-sec average impact force perpendicular to the bridge railing was approximately 276 kN (62 kips). The truck became parallel to the bridge railing at 0.180 sec. At 0.260 sec, the truck exited the bridge railing at an angle of 9.4 degrees.

Full-Scale Crash Tests

Test FSCR-1 [8,165-kg (18,000-lb), 82.4 km/hr (51.2 mph), 16.8 degrees] hit the bridge rail tip approximately 45.7 cm (1 ft 6 in.)

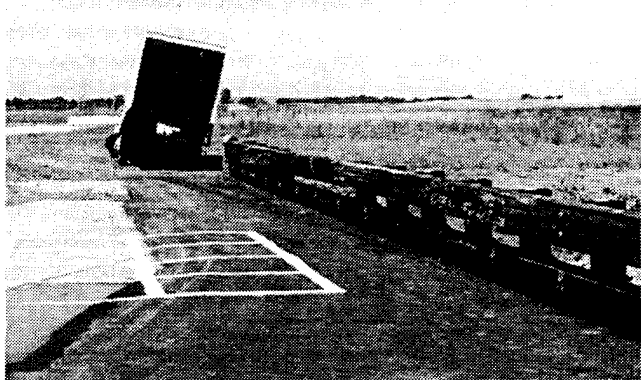


FIGURE 5 Impact location, vehicle damage, and bridge rail damage. Test FSCR-1.

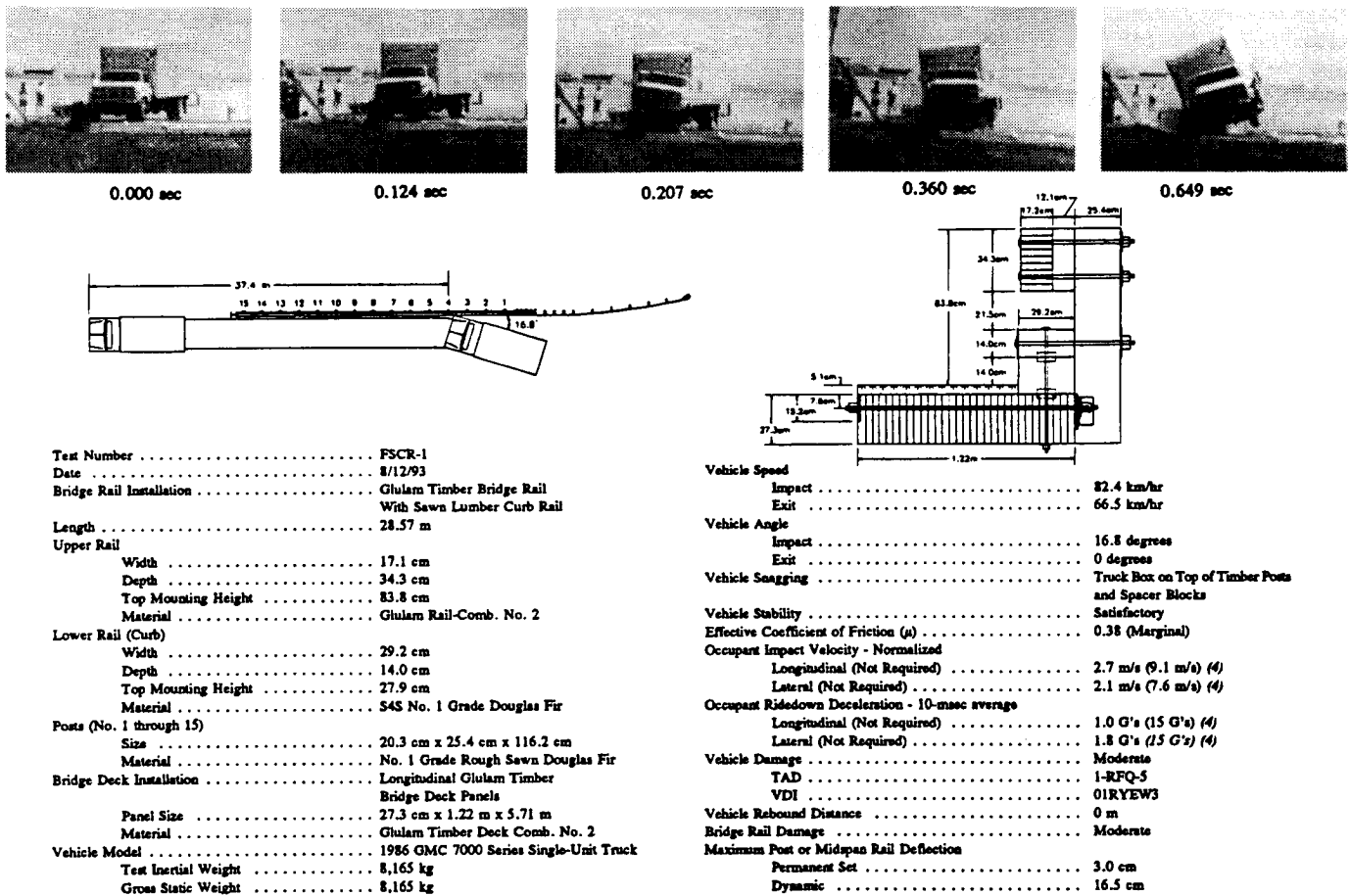


FIGURE 6 Summary of test results and sequential photographs, Test FSCR-1.

upstream from Post No. 4 (Figure 5). A summary of the test results and the sequential photographs are presented in Figure 6.

After the initial impact with the bridge rail, the right-front corner of the bumper and quarter panel crushed inward. At 0.103 sec, the maximum dynamic lateral deflections were measured at Post No. 5 and the front end of the truck cab began to lift and roll clockwise toward the rail. At 0.124 sec, the longitudinal centerline of the truck cab and box remained parallel, and at 0.145 sec. the truck box began rotating toward the rail while the truck cab began rotating away from the rail. At 0.160 sec, the right-front corner of the truck box extended over the rail and the right-front tire was crushed inward under the engine. At 0.340 sec. the truck cab began rotating toward the rail. The left-rear tire lost contact with the ground at 0.400 sec. At 0.413 sec, the truck cab was approximately parallel to the bridge rail with a velocity of 69.8 km/hr (43.4 mph). The truck box achieved a maximum roll angle of approximately 31 degrees toward the rail at 0.649 sec. At the same time, the right-rear tire also lost contact with the ground. The truck cab achieved a maximum roll angle of approximately 23 degrees toward the rail at 0.739 sec. At 1.500 sec, the truck box rolled away from the rail, and at 1.739 sec, the left-front tire contacted the ground and the vehicle exited the bridge railing at a speed of approximately 66.5 km/hr (41.3 mph) and at a 0-degree angle. The effective coefficient of friction was determined to be approximately 0.38.

Exterior vehicle damage was moderate (Figure 5). Vehicle damage occurred to several body locations, including the door and quar-

ter panels, engine hood, front bumper, right-side wheels and rims, front axle, engine hood, truck box and support frame, side-mounted foot steps, and fuel tank. The right-corner of the front bumper and the right-side door and quarter panels were crushed inward. The front axle, with attached tires and steel rims, became detached from the truck and came to rest under the left-side of the truck cab. The right-front and right-rear (outer dual) tires were deflated.

The moderate bridge railing damage near the impact area is shown in Figure 5. The downstream end of the glulam rail adjacent to Post No. 4 was fractured on the lower part of the rail. The curb rail received significant gouging between Post Nos. 4 and 5. Deep gouges and scrapes occurred to the top of the glulam rail from Post Nos. 7-14. Nine timber bridge posts, Post Nos. 7-15, were damaged during the crash test, as shown in Figure 5. The glulam timber bridge deck received some superficial surface cracks near Post No. 4. The crack width ranged between 1.6 to 3.2 mm (1/16 and 1/8 in.).

The maximum lateral permanent set deflections for midspan rail and post locations, as determined from field measurements in the impact region, were approximately 3.0 cm (1.2 in.) and 2.8 cm (1.1 in.), respectively. The maximum dynamic lateral deflections for midspan rail and post locations (determined from high-speed film analysis) were 14.5 cm (5.7 in.) and 16.5 cm (6.5 in.), respectively.

The GC-8000 bridge railing was originally designed and was to be evaluated according to the AASHTO PL2 (4) guidelines. However, following the successful 8.165-k: (18,000-lb) single-unit truck test, it was determined that the bridge railing could potentially

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meet the *NCHRP report 350 (8)* pickup truck strength test. Therefore, the 2,000-kg (4,409-lb) pickup test at 100 km/hr (62.1 mph) and 25 degrees was conducted instead of the 2,449-kg (5,400-lb) pickup test at 96.6 km/hr (60 mph) and 20 degrees.

Test FSCR-4 [2,087-kg (4,600-lb), 98.0 km/hr (60.9 mph), 24.9 degrees] impacted the bridge rail at approximately 1.76 m (5.77 ft) upstream from Post No. 8 (Figure 7). A summary of the test results and the sequential photographs are presented in Figure 8.

After the initial impact with the bridge rail, the right-front corner of the bumper and quarter panel crushed inward. At 0.054 sec, the right-front tire blew out due to contact with the sawed lumber curb rail. At 0.126 sec, maximum dynamic lateral deflections were observed at post No. 8. The entire vehicle became airborne at approximately 0.217 sec. At 0.223 sec, the pickup truck was approximately parallel to the bridge rail with a velocity of 66.5

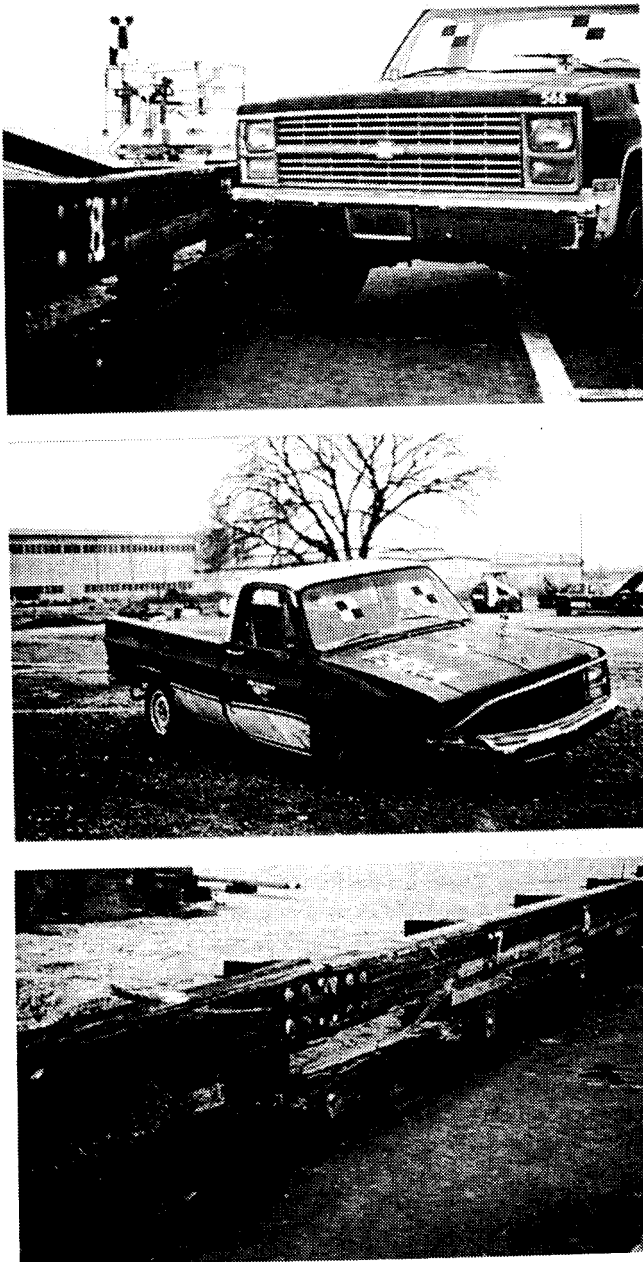


FIGURE 7 Impact location, vehicle damage, and bridge rail damage, Test FSCR-4

km/hr (41.3 mph) with a slight roll angle toward the bridge rail. At 0.418 sec, the vehicle exited the bridge railing at a speed of approximately 62.9 km/hr (39.1 mph) and angle of 10.4 degrees. The vehicle's right-front tire contacted the ground at 0.512 sec, and its left-front tire contacted the ground at 0.620 sec. The effective coefficient of friction was determined to be approximately 0.54.

Exterior vehicle damage was moderate (see Figure 7). Vehicle damage occurred to several body locations, including the door and quarter panels, front bumper, right-side tires and rims, rear bumper, engine mount, and interior floorboard. The right-front tire, was deflated and partly removed from the rim. In addition, the right-front tire, rim, and attached steering mechanism were pushed backward, and the right-side engine mount was deformed toward the engine. Interior vehicle deformation to the occupant compartment was not sufficient to cause injury to the vehicle occupants.

The minor bridge railing damage downstream from the impact location is shown in Figure 7. Scrapes and gouging occurred to the upper glulam timber and sawed lumber curb rails. Significant tire and rim contact on the curb rail was evident from the downstream side of Post No. 7 to the downstream side of Post No. 8. Longitudinal cracking occurred toward the bottom traffic-side face of the glulam rail at Post No. 8. The downstream-side of the glulam rail splice located at Post No. 8 was fractured. The flexural failure occurred in the tension region of the glulam rail (or the backside of the vertical saw-cut section) and near the downstream end of the steel splice plate. No physical damage occurred to the timber bridge posts or spacer blocks. Additional curb rail damage consisted of cracking along a vertical plane through the longitudinal centerline of the bolts. The glulam timber bridge deck received some superficial surface cracks. The crack width ranged between 1.6 and 6.4 mm (1/16 and 1/4 in.).

The maximum lateral permanent set deflections for midspan rail and post locations (determined from field measurements in the impact region) were approximately 5.3 cm (2.1 in.) and 4.8 cm (1.9 in.), respectively. The maximum dynamic lateral deflections for midspan rail and post locations were 29.2 cm (11.5 in.) and 36.1 cm (14.2 in.), respectively.

Tests FSCR-1 and FSCR-4 were evaluated according to the AASHTO PL2 and NCHRP 350 TL4 criteria. The GC-8000 bridge rail contained and smoothly redirected the test vehicles. The test vehicles did not penetrate, underide, or override the bridge railing, although controlled lateral deflection of the bridge rail is acceptable. There were no detached elements, fragments, or other debris from the bridge railing that showed potential for penetrating the occupant compartment or that presented undue hazard to other traffic. Deformations of, or intrusions into, the occupant compartment that could cause serious injuries did not occur. For Tests FSCR-1 and FSCR-4, the effective coefficients of friction were marginal [$\mu = 0.38$ and $\mu = 0.54$ ($\mu > 0.35$)]. The test vehicles remained upright during and after collision. The occupant risk values for occupant impact velocities and ridedown decelerations were satisfactory. The vehicle trajectories revealed no intrusion into adjacent traffic lanes. For Tests FSCR-1 and FSCR-4, the vehicle exit angles of 0 and 10.4 degrees, respectively, were less than 60 percent of the impact angles of 15 and 25 degrees.

CONCLUSIONS AND RECOMMENDATIONS

The safety performance evaluations of an AASHTO PL2 three-beam with channel (TBC-8000) rail and an *NCHRP Report No. 350*

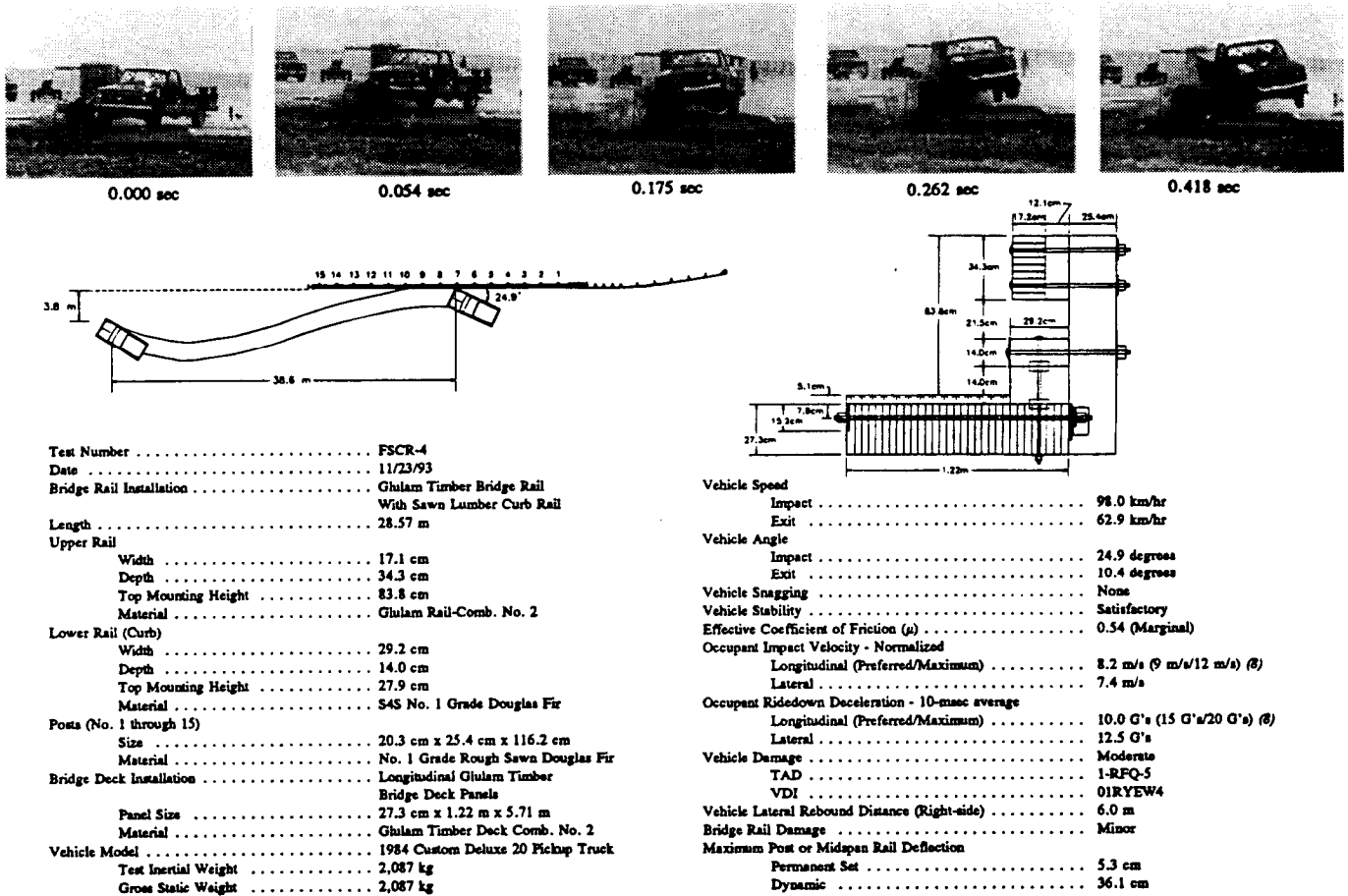


FIGURE 8 Summary of test results and sequential photographs, Test FSCR-4.

TL4 glulam rail with curb (GC-8000) were tested according to the applicable guidelines, and both were acceptable. The result is two new crashworthy bridge railings that are recommended for use on longitudinal timber bridges. Although the two rails were tested on a longitudinal glulam timber bridge deck, both could be adapted for use on other longitudinal timber bridge decks.

The development of the TBC-8000 bridge railing satisfied the concern for economy while also providing a crashworthy bridge railing system for timber bridge decks on higher performance roadways. Although both railings performed similarly according to the evaluation factors of structural adequacy, occupant risk and vehicle trajectory, the vehicle damage to the 8,000-kg (17,637-lb) single-unit truck was more extensive for the GC-8000 impact, and its repair costs also would be higher.

The TBC-8000 was easy to install; therefore it should have low construction costs. The material cost for the TBC-8000 was approximately \$174/m (\$53/ft). The glulam curb system (GC-8000) is aesthetically pleasing but more expensive than the three-beam with channel (TBC-8000) system. The material cost for the GC-8000 was approximately \$354/m (\$108/ft).

Further testing should be conducted if it is deemed necessary that both transitions and the TBC-8000 railing meet NCHRP Report 350 TL4 criteria. Further testing will be required because no 8,000-kg (17,637-lb) single-unit truck test or 2,000-kg (4,409-lb) pickup truck test was conducted on either transition. In addition, the TL4

pickup truck test has the potential for significant occupant compartment deformation and could cause the TBC-8000 railing to fail the NCHRP Report 350 TL-4 crash standards.

ACKNOWLEDGMENTS

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Two Test Level 4 Bridge Railing and Transition Systems for Transverse Timber Deck Bridges

Ronald K. Faller, Michael A. Ritter, Barry T. Rosson, Michael D. Fowler, and Sheila R. Duwadi

The Midwest Roadside Safety Facility, in cooperation with the Forest Products Laboratory, which is part of the U.S. Department of Agriculture's Forest Service, and FHWA, designed two bridge railing and approach guardrail transition systems for use on bridges with transverse glue-laminated timber decks. The bridge railing and transition systems were developed and crash tested for use on higher-service-level roadways and evaluated according to the Test Level 4 safety performance criteria presented in *NCHRP Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features*. The first railing system was constructed with glulam timber components, whereas the second railing system was configured with steel hardware. Eight full-scale crash tests were performed, and the bridge railing and transition systems were acceptable according to current safety standards.

Over the past 30 years, numerous bridge railing systems have been developed and evaluated according to established vehicular crash-testing standards. Most of the bridge railings previously crash tested have consisted of concrete, steel, and aluminum railings attached to concrete bridge decks. It is well known that a growing number of timber bridges with transverse and longitudinal timber decks are being constructed throughout the country. Therefore, the demand for crash-worthy railing systems has become more evident with the increasing use of timber decks located on secondary highways, county roads, and local roads. Over the past 10 years, several crash-worthy bridge railing systems have been developed for use on longitudinal timber decks. In addition, these railing systems were developed for multiple service levels that ranged from low-speed, low-volume roads to higher-service-level roadways. However, little research has been conducted in the development of crash-worthy railing systems for bridges with transverse timber decks, and those that have been developed are for use on low-to-medium service-level roadways. For timber to be a viable and economical alternative in the construction of transverse timber decks, additional railing systems must be developed and crash tested for timber decks that are located on higher-service-level roadways for which no railing systems existed before.

Because of the need to develop bridge railing systems for this higher service level, the Midwest Roadside Safety Facility (MwRSF),

in cooperation with the Forest Products Laboratory (FPL), which is a part of the U.S. Department of Agriculture's Forest Service, and FHWA, undertook the task of developing two higher-service-level bridge railings and approach guardrail transitions.

RESEARCH OBJECTIVES

The primary objective of this research project was to develop and evaluate two bridge railings and approach guardrail transitions for use with transverse glue-laminated (glulam) timber deck bridges that were located on higher-service-level roadways. The bridge railing and transition systems were developed to meet Test Level 4 (TL-4) evaluation criteria that are described in *NCHRP Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features (1)*.

The first bridge railing, referred to as System No. 1, was a wood system that was constructed with an upper rail, a lower curb rail, scupper blocks, posts, and blockouts, all of which were manufactured from glulam timber. Photographs of the railing system of the wood bridge and the attached approach guardrail transition are shown in Figure 1. The second bridge railing, referred to as System No. 2, was a steel system that was constructed with a three-beam rail, an upper structural tube rail, and wide flange posts and blockouts. Photographs of the steel bridge railing system and the attached approach guardrail transition are provided in Figure 2.

Another objective of the research project was to determine the actual forces imparted to the key components of the bridge railing systems. Knowledge of these force levels would allow researchers and engineers to make minor modifications to the crash-tested designs without additional full-scale crash testing and would provide insight into the design of future systems.

RESEARCH PLAN

The research objectives were accomplished with the successful completion of several tasks. First, a literature search was performed to review the previously developed, high-performance-level bridge railing systems, as well as to review bridge railings that were developed for timber deck bridges. The review was deemed necessary because it was envisioned that the two new bridge railing designs would likely use technologies and design details from existing crash-worthy railing systems. Second, bridge railing concepts were prepared so that an analysis and design phase could be performed on all structural members and connections.

R. K. Faller, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, 1901 Y Street, Building C, Lincoln, NE 68588-0601. M. A. Ritter, USDA Forest Service, Forest Products Laboratory, One Gifford Pinchot Drive, Madison, WI 53705. B. T. Rosson, Department of Civil Engineering, University of Nebraska-Lincoln, W348 Nebraska Hall, Lincoln, NE 68588-0531. M. D. Fowler, MK Centennial, 15000 West 64th Avenue, P.O. Drawer 1307, Arvada, CO 80001. S. R. Duwadi, Turner-Fairbank Highway Research Center, Federal Highway Administration, 6300 Georgetown Pike, McLean, VA 22101-2296.



FIGURE 1 System No. 1: glulam rail with curb bridge railing (*top*) and thrie beam with curb transition (*bottom*).

Subsequently, computer simulation modeling was conducted by using BARRIER VII to aid in the analysis and design of the bridge railing and approach guardrail transition systems (2). For the wood system, static component testing was then performed on selected bridge components to obtain (a) static stiffness properties for use in the calibration of the computer simulation modeling and (b) calibration factors for instrumentation sensors that were located in strategically placed structural components. Additional instrumentation was placed on the bridge railing systems to help determine the actual dynamic loads imparted into the bridge railing and deck systems. The researchers deemed that the dynamic load information was necessary because additional economy could be provided with the downsizing of specific structural components.

Next, eight full-scale vehicle crash tests (two crash tests on each bridge railing and transition system) were performed by using $\frac{3}{4}$ -ton pickup trucks and single-unit trucks. Test results were analyzed, evaluated, and documented. Conclusions and recommendations that pertained to the safety performance of each bridge railing and transition system were then made.

BRIDGE RAILING HISTORY

The primary purpose of a bridge railing is to safely contain errant vehicles and prevent them from falling off the bridge. Therefore, railings must be designed to withstand the force of a striking vehicle with-

out endangering its occupants. In designing railing systems for highway bridges, engineers have traditionally assumed that vehicle impact forces can be approximated by equivalent static loads that are applied to railing elements. Until recently, criteria presented in AASHTO's *Standard Specifications for Highway Bridges* (3) required that bridge railings be designed to resist an outward transverse static load of 44.5 kN. Despite the widespread use of design requirements that is primarily based on static load criteria, the need for more appropriate criteria that covers full-scale vehicle crash tests has long been recognized. The first set of U.S. guidelines for full-scale vehicle crash tests was published in 1962 (4). In 1981, *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances* was published (5). This comprehensive report provided recommendations that were relative to crash testing and an evaluation of longitudinal barriers. It also served as the basis for requirements for future bridge rail crash testing.

The first recognition of full-scale crash tests in a national bridge specification was in 1989 after AASHTO published *Guide Specifications for Bridge Railings* (6). This specification presents recommendations for the development, testing, and use of crash-tested bridge railings and refers extensively to *NCHRP Report 230* for crash-test procedures and requirements. For this specification, recommended requirements for rail tests were based on three performance levels: Performance Level 1 (PL-1), PL-2, and PL-3. PL-1 requirements represent the "weakest" system, and PL-3 represents the "strongest" system. The recently published *NCHRP Report 350* identifies six test



FIGURE 2 System No. 2: steel thrie beam with tube bridge railing (*top*) and thrie beam with tube transition (*bottom*).

levels for evaluating longitudinal barriers—Test Level 1 (TL-1) through TL-6. Although this document does not include objective criteria for relating a test level to a specific roadway type, the lower test levels are generally intended for use on lower-service-level roadways and on certain types of work zones, whereas the higher test levels are intended for use on higher-service-level roadways.

In 1994, AASHTO published the *AASHTO LRFD Bridge Design Specifications* (7) as an update to the *Standard Specifications for Highway Bridges* (3) and to the *Guide Specifications for Bridge Railings* (6). For crash testing bridge railings, three performance levels were provided, and guidelines followed procedures that were presented in both the *AASHTO Guide Specifications for Bridge Railings* and *NCHRP Report 350*. Yield line and inelastic analysis and design procedures, as originally developed by Hirsch (8), were also provided for bridge railings as a replacement for the 44.5-kN equivalent static load procedures.

Emphasis on the use of crash-tested rails for new federally funded projects has significantly increased the role of full-scale crash tests as a means of evaluating railing performance. Recently, FHWA officially adopted *NCHRP Report 350* as a replacement for *NCHRP Report 230* and has strongly suggested that AASHTO also adopt the test-level definitions presented in *NCHRP Report 350*, thus making crash-tested railings mandatory for most bridges. Most highways on which wood bridges are installed will require railings that meet the *NCHRP Report 350* requirements for TL-1 through TL-4.

As of August 1986, 22 bridge rails had been successfully crash tested in accordance with the guidelines specified in *NCHRP Report 230* and approved for use in federal aid projects by FHWA (9). By August 1990, 25 additional bridge rails had been successfully crash tested in accordance with the requirements of AASHTO's *Guide Specifications for Bridge Railings* and also approved by FHWA for use in federal aid projects (10). Of these crash-tested railings, 46 were used on concrete bridge decks, and only one was used on a wood deck (11).

During the 1990s, two other research programs led to the development of crash-worthy railing systems for timber deck bridges. The first program, a collaborative effort between MwRSF, FPL, and FHWA engineers, resulted in the development of nine railing systems for longitudinal timber deck bridges (12–17). Simultaneously with the MwRSF research program, researchers at West Virginia University conducted a research effort to develop three AASHTO PL-1 railing systems for transverse wood decks (18).

TEST REQUIREMENTS AND EVALUATION CRITERIA

According to the TL-4 criteria presented in *NCHRP Report 350*, longitudinal barriers must be subjected to three full-scale vehicle crash tests: (a) a small car weighing 820 kg colliding at a speed of 100 km/h

and at an angle of 20 degrees, (b) a pickup truck weighing 2000 kg colliding at a speed of 100 km/h and at an angle of 25 degrees, and (c) a single-unit truck weighing 8000 kg colliding at a speed of 80 km/h and at an angle of 15 degrees. For this research project, crash tests were performed by using only the pickup truck and single-unit truck impact conditions. Although the small car test is used to evaluate the overall performance of the length-of-need section and to assess occupant-risk problems that arise from snagging or overturning the vehicle, it was deemed unnecessary for several reasons.

First, during the design of both barrier systems, special attention was given to prevent geometric incompatibilities that would cause the small car tests to fail as a result of excessive snagging or overturning. Second, the structural adequacy of higher-service-level barrier systems is not a concern for the small car test because of the relatively minor impact severity as compared with the impact severity for the pickup truck and the single-unit truck impact conditions. The impact severity for the pickup truck test is about 270 percent greater than the impact severity provided by the small car test. Third, a small car crash test was successfully conducted on a similar wood bridge railing system by the Southwest Research Institute (11). Finally, three-beam barriers struck by small cars have been shown to meet safety performance standards and to be essentially rigid (19–21), with no significant potential for occupant-risk problems that arise from snagging or overturning. For these reasons, the small car crash test was considered unnecessary for the systems that were developed under this research project.

Evaluation criteria for full-scale crash tests are based on three appraisal areas: (a) structural adequacy, (b) occupant risk, and (c) vehicle trajectory after collision. Criteria for structural adequacy are intended to evaluate the ability of the railing to contain, redirect, or allow controlled vehicle penetration in a predictable manner. Occupant risk evaluates the degree of hazard to occupants of the striking vehicle. Vehicle trajectory after collision is concerned with the path and final position of the striking vehicle and the probable involvement of this vehicle in secondary collisions. Note that these criteria address only the safety and dynamic performance of the barrier and do not include service criteria such as aesthetics, economics, bridge damage, or postimpact maintenance requirements. The evaluation criteria are summarized in *NCHRP Report 350*.

DEVELOPMENT PHASE

Transverse Panels

Highway bridges with transverse timber decks and those that require crash-tested railing systems are most commonly constructed with glulam timber deck panels. Transverse glulam timber decks are constructed of panels that are oriented with the lumber length perpendicular to the direction of traffic. Individual lumber laminations are placed edgewise and are glued together with waterproof structural adhesives. These panels are typically 1.22 m wide and 127 to 171 mm thick and effectively act as a thin plate. To form the bridge deck, panels are placed side by side and are supported by longitudinal glulam or steel beams. These longitudinal beams are designed to carry the vertical loads and are braced by either glulam or steel diaphragms so as to provide lateral stiffness to the bridge structure. Given that the panel orientation is perpendicular to traffic, railing loads primarily introduce tension and bending in the panels parallel to the wood grain. Unlike the longitudinal glulam timber decks, tension that is perpendicular to the wood grain is not a primary design consideration.

Bridge Rail Design

The primary emphasis of the railing design process was to develop rails that would meet the requirements of *NCHRP Report 350*. In addition, it was determined that consideration should be given to

- Extent of probable damage to the structure after vehicle impact and the difficulty and cost of required repairs;
- Adaptability of the railing to different types of wood decks;
- Cost of the rail system to the user, including material, fabrication, and construction;
- Ease of railing construction and maintenance; and
- Aesthetics of the rail system.

The development phase concluded with the design of several railing and transition systems and the preparation of plans and specifications for testing. The selection and design of these final systems were based on a review of other railings that had been successfully crash tested, as well as those railings that are currently used on wood bridges but have not been crash tested. To the extent possible, feasible designs were evaluated by using BARRIER VII computer simulation modeling (2). Although several proven computer models were used, it was sometimes difficult to adapt the programs for wood components because the behavior and properties of the wood systems at ultimate loading were unknown. For the wood railing system, static component testing was conducted to obtain stiffness properties for use in the simulation modeling and to determine calibration factors for selected instrumentation sensors. Details of this testing can be found in Fowler's master's thesis (22).

TEST BRIDGE

Testing of the bridge railing and approach guardrail transition systems was conducted at MwRSF's outdoor test site in Lincoln, Nebraska. To perform all the barrier testing, a full-sized test bridge was constructed. The test bridge measured about 3.96 m wide and 36.58 m long and consisted of three simply supported spans that measured about 12.19 m each.

The transverse deck system was constructed of 130-mm-thick by 1.22-m-wide glulam timber panels. The glulam timber for the deck was Combination No. 47 Southern Yellow Pine (SYP), as specified in *AASHTO LRFD Bridge Design Specifications* (7). The timber was also treated according to the American Wood Preservers' Association (AWPA) Standard C14 (23). Thirty glulam timber panels were placed side by side to achieve the 36.58-m length and were attached to the longitudinal glulam beams with standard aluminum deck brackets.

The test bridge was positioned on concrete supports that were placed in a 2.13-m-deep excavated test pit. The concrete supports were placed so that the top of the test bridge was 51 mm below the concrete surface to allow for placement of the bridge deck-wearing surface. A detailed discussion of the test bridge is beyond the scope of this paper and is presented in detail by Fowler (22).

SYSTEM NO. 1: WOOD RAILING

Design Details

The first bridge railing system was designed for an all-wood system, except for the structural steel connections. The system was

constructed with an upper rail, a lower curb rail, scupper blocks, bridge posts, and rail blockouts. Specific details of the system are provided in Figure 3. For the wood system, glulam timber for the upper rail and post members was Combination No. 48 SYP, as specified in *AASHTO LRFD Bridge Design Specifications* (7), and was treated with pentachlorophenol in heavy oil according to AWWA Standard C14 requirements (23). Glulam timber for the curbs, scuppers, and spacer blocks were fabricated with Combination No. 47 SYP, as specified by AASHTO, and treated in the same manner as described previously according to AWWA Standard C14.

System No. 1 was configured similarly to the PL-1 and TL-4 glulam timber rail with curb systems previously developed for longitudinal decks (12,13,15,16). However, for this system, all wood components were fabricated from glulam timber, whereas the previous systems used glulam and sawed lumber. In addition, all structural members, as well as the steel hardware, were resized to account for the increased post spacing from 1905 to 2438 mm. The new post spacing was selected to optimize the design and significantly improve the constructability of the railing system, which was based on 1219-mm-wide deck panels.

A transition system using a TL-4 approach guardrail was designed for attachment to each end of the bridge railing system. The system was constructed with a steel thrie-beam upper rail, a lower curb rail, guardrail posts, rail blockouts, and special transition blocks and connectors. Specific details of the approach guardrail transition that is used with System No. 1 are provided in Figure 4.

Bridge Rail Crash Tests

The wood bridge railing system was subjected to two full-scale vehicle crash tests. Details of crash tests are provided in this section. It is noted that instrumentation sensors were strategically placed on selected bridge railing components. However, a detailed discussion of the instrumentation results is beyond the scope of this paper but is presented in detail by Fowler (22).

The first crash test, Test TRBR-1, was successfully performed with a 1986 Ford F-800 Series, single-unit truck with a test inertial mass of 8000 kg and at impact conditions of 74.8 km/h and at an angle of 16 degrees. During impact, the vehicle exited the railing system at a speed of 47.3 km/h and at an angle of 0 degrees. The maximum lateral permanent set deflection and the dynamic rail deflection were observed to be 10 and 84 mm, respectively. The location of vehicle impact with the bridge railing, vehicle damage, and barrier damage are shown in Figure 5.

The second crash test, Test TRBR-2, was successfully performed with a 1988 Ford F-250, ¾-ton pickup truck with a test inertial mass of 1993 kg and at impact conditions of 99.2 km/h and at an angle of 27.4 degrees. During impact, the vehicle exited the railing system at a speed of 62.3 km/h and at an angle of 2.1 degrees. The maximum lateral permanent set deflection and the dynamic rail deflection were observed to be 29 and 203 mm, respectively. The location of the vehicle impact with the bridge railing, vehicle damage, and barrier damage are shown in Figure 6.

Following an analysis of the test results, it was determined that the wood bridge railing system met the TL-4 safety performance criteria presented in *NCHRP Report 350* (J). No significant damage to the test bridge was evident from the vehicle impact tests. For the bridge railing system, damage consisted primarily of rail gouging and scraping. All glulam timber railings remained intact and serviceable after the tests, and replacement of the railing was not considered necessary.

Transition Crash Tests

The approach guardrail transition that is used with the wood bridge railing system was also subjected to two full-scale vehicle crash tests. Details of crash tests are provided in this section.

The first crash test, Test TRBR-3, was successfully performed with a 1987 Ford F-250, ¾-ton pickup truck with a test inertial mass of 2029 kg and at impact conditions of 104.9 km/h and at an angle of 26.4 degrees. During impact, the vehicle exited the transition system at a speed of 71.1 km/h and at an angle of 11.9 degrees. The maximum lateral permanent set deflection and the dynamic rail deflection were observed to be 35 and 163 mm, respectively. The location of vehicle impact with the approach guardrail transition, vehicle damage, and barrier damage are shown in Figure 7.

The second crash test, Test TRBR-4, was successfully performed with a 1988 Ford F-700 Series, single-unit truck with a test inertial mass of 8003 kg and at impact conditions of 82.5 km/h and at an angle of 13.7 degrees. During impact, the vehicle exited the transition system at a speed of 25.3 km/h and at an angle of less than 1 degree. The maximum lateral permanent set deflection and the dynamic rail deflection were observed to be 49 and 124 mm, respectively. The location of vehicle impact with the approach guardrail transition, vehicle damage, and barrier damage are shown in Figure 8.

During the impact event, a failure occurred in the connection hardware between the truck box and the steel frame that caused the box to release from the frame and travel over the bridge railing. From an analysis of the high-speed photographs, it was evident that this failure occurred after the truck had reached the bridge railing region and was not a result from any specific contact with components of the approach guardrail transition. Because a single-unit truck had successfully performed on the bridge railing system and no vehicle snagging had occurred in the transition region, the researchers determined that a retest was not required. Further investigation revealed that the release of the truck box resulted from an inadequate number and size of steel connection hardware.

After analyzing the test results, it was determined that the approach guardrail transition that is used with the wood bridge railing system met the TL-4 safety performance criteria presented in *NCHRP Report 350*. No significant damage to the test bridge was evident from the vehicle impact tests. For the approach guardrail transition system, damage consisted primarily of a deformed thrie-beam rail, displaced guardrail posts, and gouged and scraped glulam rail and thrie-beam blockouts. All glulam timber railings remained intact and serviceable after the tests, whereas the steel thrie beam required replacement in the vicinity of impact after each crash test.

SYSTEM NO. 2: STEEL RAILING

Design Details

The second bridge railing system was designed as an all-steel system. This system was constructed with a thrie-beam rail, an upper structural tube rail, wide flange bridge posts and rail blockouts, and deck-mounting plates. Specific details of this system are provided in Figure 9. For the steel system, a 10-gauge, thrie-beam rail was blocked from the wide flange posts with wide flange spacers. A structural tube rail was then attached to the top of the spacer blocks. The lower end of each post was bolted to two steel plates that were connected to the top and bottom surfaces of the bridge deck with vertical bolts.

Attachment 3

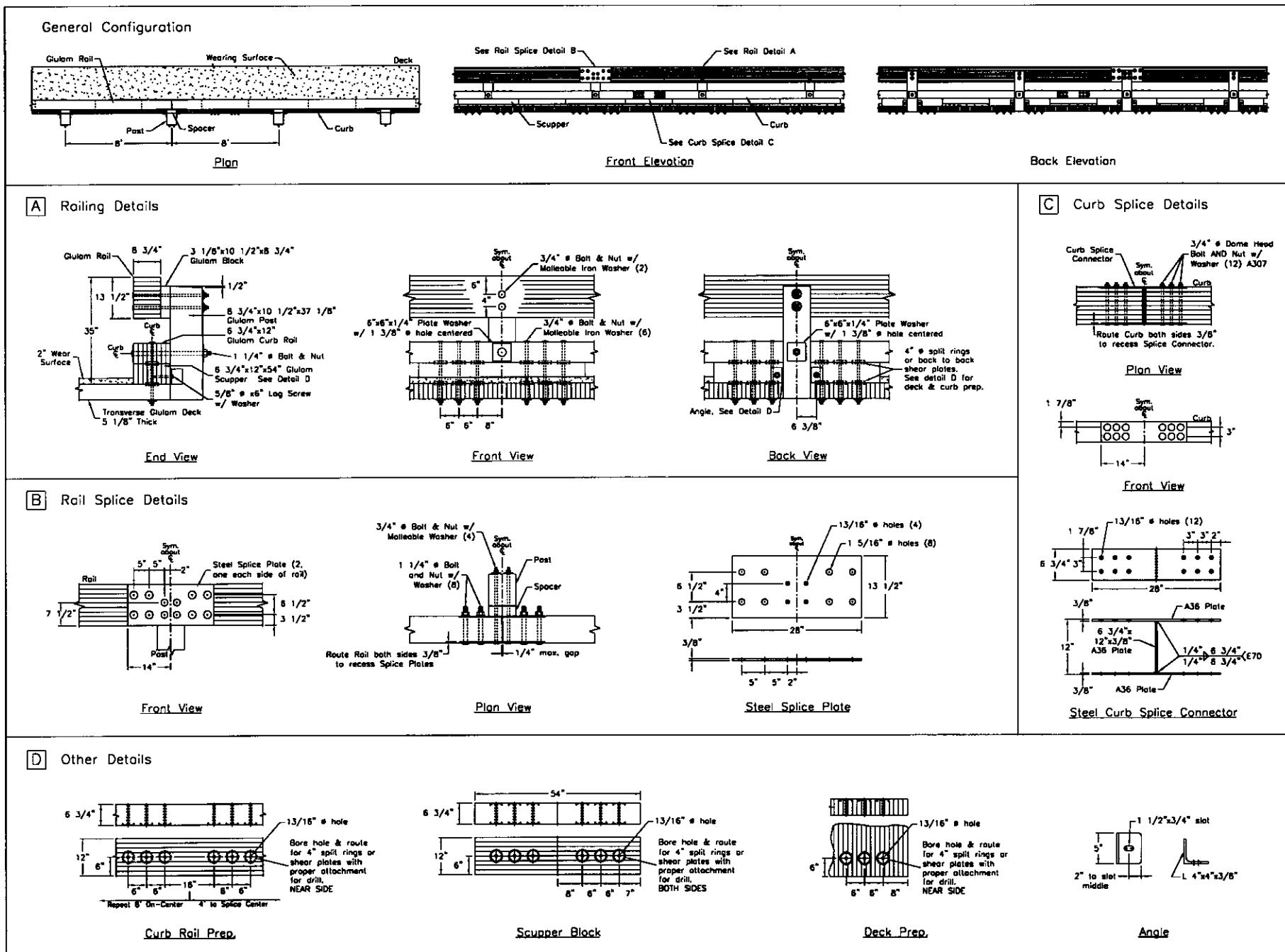


FIGURE 3 System No. 1: wood bridge railing design details (1 in. = 25.4 mm).

Attachment 3

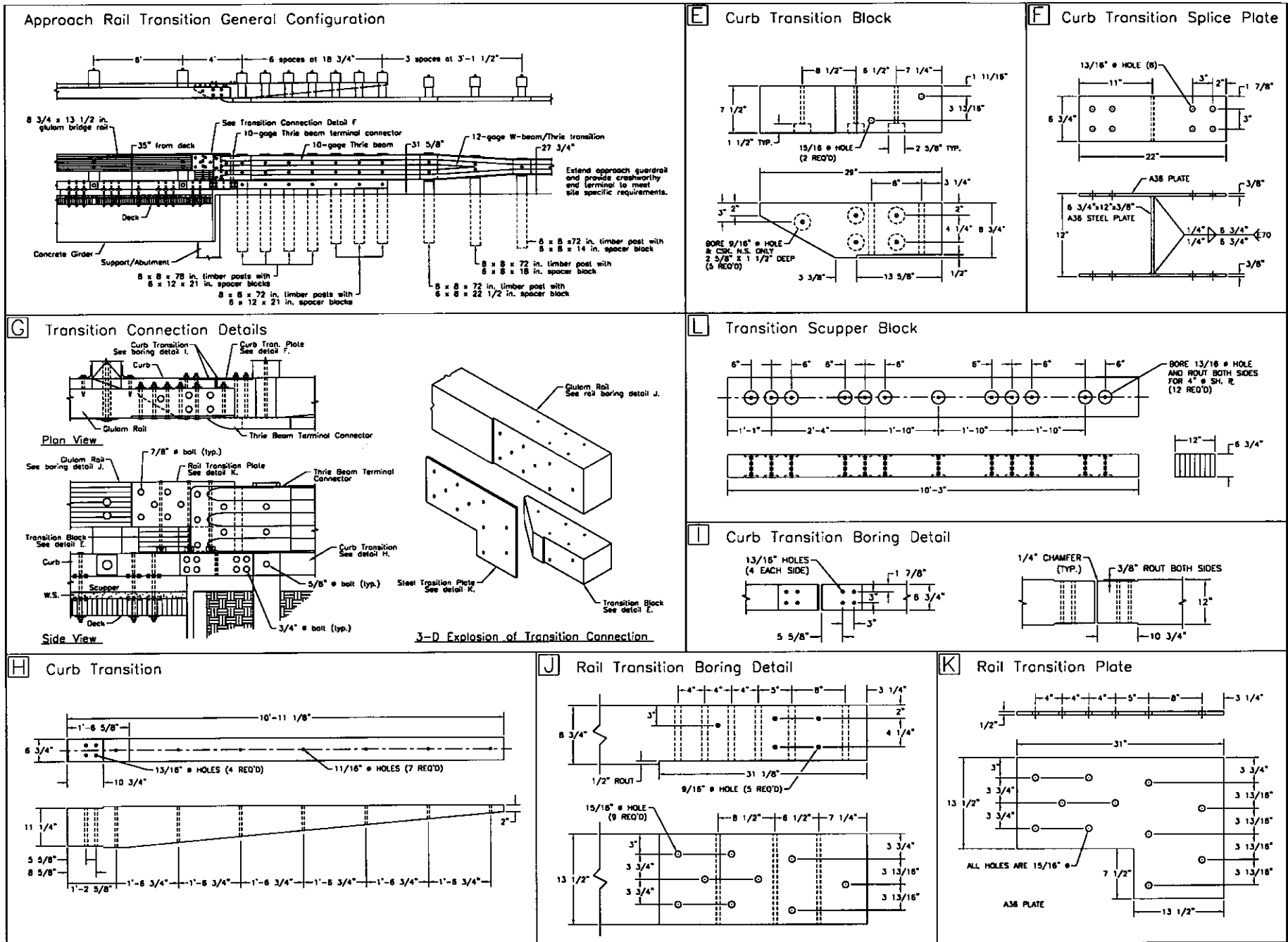


FIGURE 4 System No. 1: approach guardrail transition design details (1 in. = 25.4 mm).

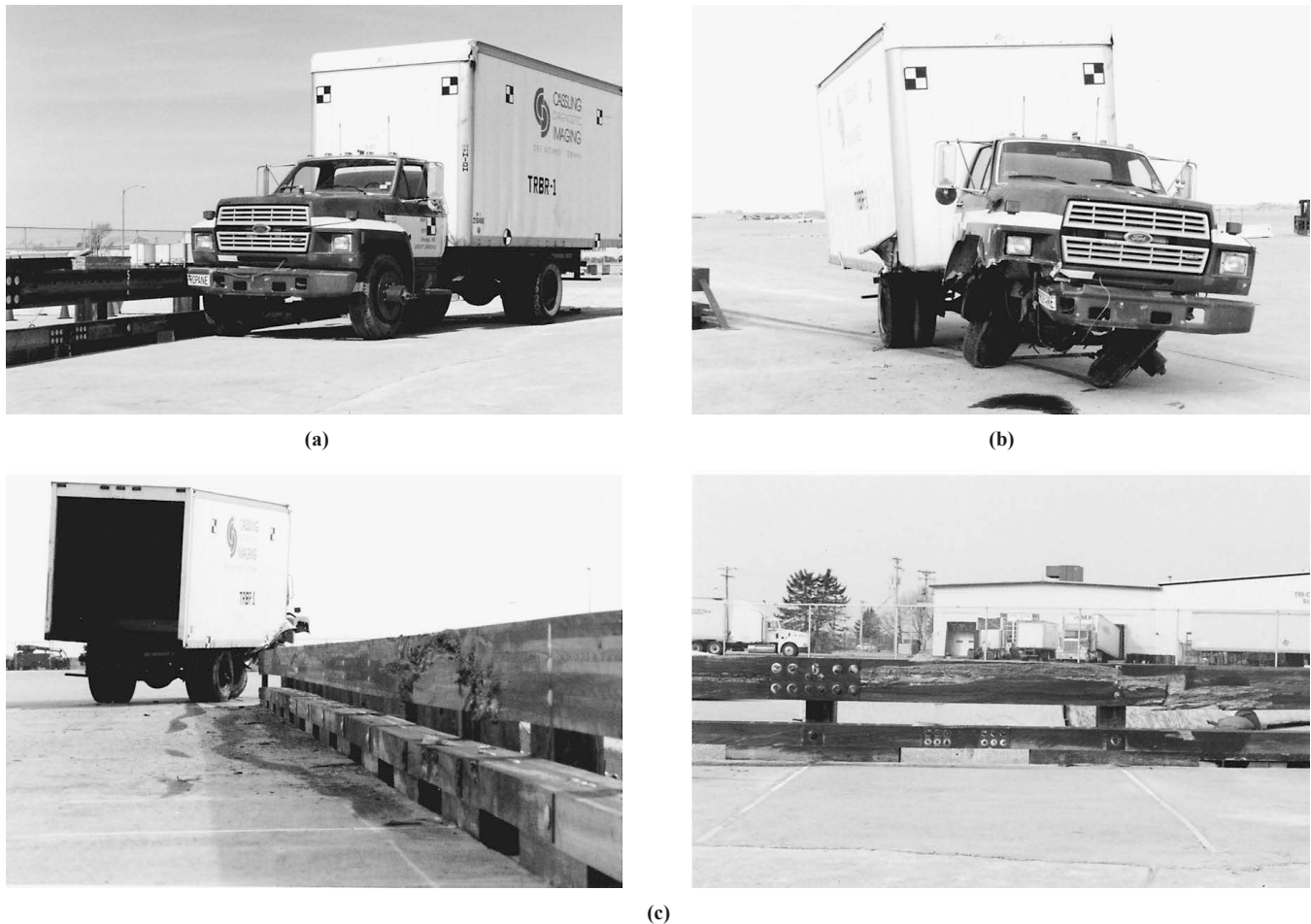


FIGURE 5 Test TRBR-1: (a) impact location, (b) vehicle damage, and (c) bridge railing damage.

System No. 2 was configured similarly to the PL-2 steel thrie beam and channel bridge railing system that was developed for longitudinal decks (13,15–16). However, for this system a structural tube member was used for the upper rail instead of using a channel section to account for the increased post spacing from 1905 to 2438 mm. The change was made to provide greater load distribution and increased resistance to lateral buckling of the upper rail.

A transition system that uses a TL-4 approach guardrail was designed for attachment to each end of the bridge railing system. The system was constructed with a steel thrie-beam rail, a sloped structural tube end rail, guardrail posts, and rail blockouts. Specific details of the approach guardrail transition that is used with System No. 2 are provided in Figure 10.

Bridge Rail Crash Tests

The steel bridge railing system was subjected to two full-scale vehicle crash tests. Details of the crash tests are provided in this section. Once again, instrumentation sensors were strategically placed on selected bridge railing components. A detailed discussion of the instrumentation results is beyond the scope of this paper and will be provided in future publications.

The first crash test, Test STTR-1, was successfully performed with a 1990 Ford F-250, $\frac{3}{4}$ -ton pickup truck with a test inertial mass of 1994 kg and at impact conditions of 93.7 km/h and at an angle of 25.5 degrees. During impact, the vehicle exited the railing system at a speed of 62.3 km/h and at an angle of 1.5 degrees. The maximum lateral permanent set deflection and the dynamic rail deflection were observed to be 92 and 137 mm, respectively. The location of vehicle impact with the bridge railing, vehicle damage, and barrier damage are shown in Figure 11.

The second crash test, Test STTR-2, was successfully performed with a 1985 Ford F-800 Series, single-unit truck with a test inertial mass of 8067 kg and at impact conditions of 76.4 km/h and at angle of 14.6 degrees. During the impact, the vehicle exited the railing system at a speed of 63.6 km/h and at an angle of less than 1 degree. The deflection of the maximum lateral permanent set rail was observed to be 136 mm. The location of the vehicle impact with the bridge railing, vehicle damage, and barrier damage are shown in Figure 12.

After analyzing the test results, it was determined that the steel bridge railing system met the TL-4 safety performance criteria presented in *NCHRP Report 350*. No significant damage to the test bridge was evident from the vehicle impact tests. For the bridge railing system, damage consisted primarily of permanent deformation of the thrie-beam rail, tube rail, wide flange posts, and rail spacers. Although all of the steel members remained intact and serviceable

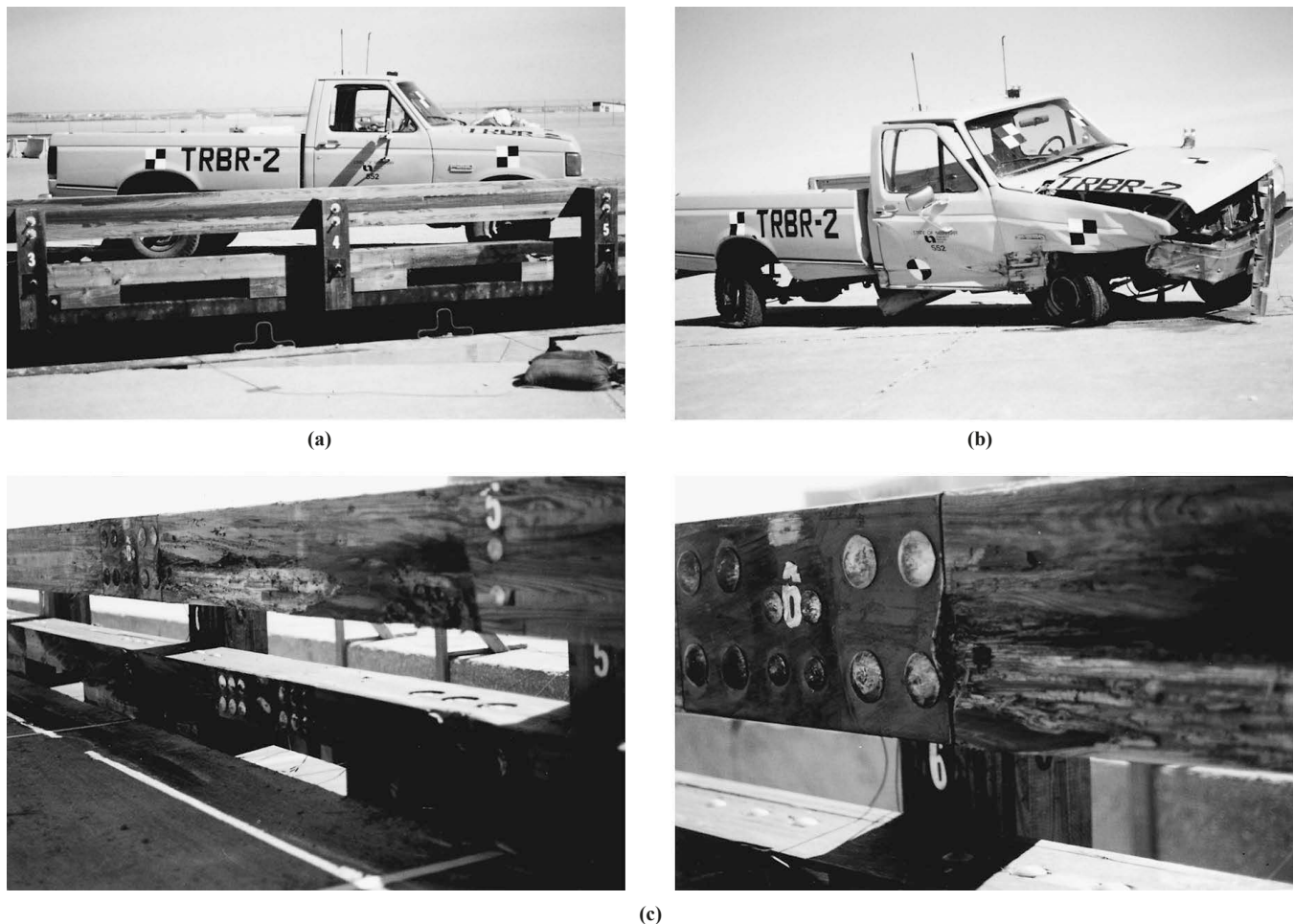


FIGURE 6 Test TRBR-2: (a) impact location, (b) vehicle damage, and (c) bridge railing damage.

after the tests, steel members with visual permanent set deformations required replacement in the vicinity of the impact after each crash test.

Transition Crash Tests

The approach guardrail transition that is used with the steel bridge railing system was also subjected to two full-scale vehicle crash tests. Details of the crash tests are provided in this section.

The first crash test, Test STTR-3, was successfully performed with a 1988 Ford F-250, $\frac{3}{4}$ -ton pickup truck with a test inertial mass of 1997 kg and at impact conditions of 101 km/h and at an angle of 25.6 degrees. During impact, the vehicle exited the transition system at a speed of 73.5 km/h and at an angle of 4.9 degrees. The maximum lateral permanent set deflection and the dynamic rail deflection were observed to be 67 and 143 mm, respectively. The location of vehicle impact with the approach guardrail transition, vehicle damage, and barrier damage are shown in Figure 13.

The second crash test, Test STTR-4, was successfully performed with a 1988 Chevrolet C60, single-unit truck with a test inertial mass of 8006 kg and at impact conditions of 81.8 km/h and at an angle of 15.2 degrees. During impact, the vehicle exited the transition system at a speed of 65.2 km/h and at an angle of 7.8 degrees. The maximum

lateral permanent set deflection and the dynamic rail deflection were observed to be 38 and 93 mm, respectively. The location of vehicle impact with the approach guardrail transition, vehicle damage, and barrier damage are shown in Figure 14.

After analyzing the test results, it was determined that the approach guardrail transition that was used with the steel bridge railing system met the TL-4 safety performance criteria presented in *NCHRP Report 350*. No significant damage to the test bridge was evident from the vehicle impact tests. For the approach guardrail transition system, damage consisted primarily of deformed three-beam rail and bridge posts and displaced guardrail posts. Although all of the steel members remained intact and serviceable after the tests, steel members with visual permanent set deformations required replacement in the vicinity of the impact after each crash test.

DISCUSSION OF RESULTS AND RECOMMENDATIONS

As stated previously, the researchers installed instrumentation sensors on key components of the railing systems in an attempt to measure the actual forces imparted into the timber deck. For the wood system, the test results revealed that the bridge railing per-

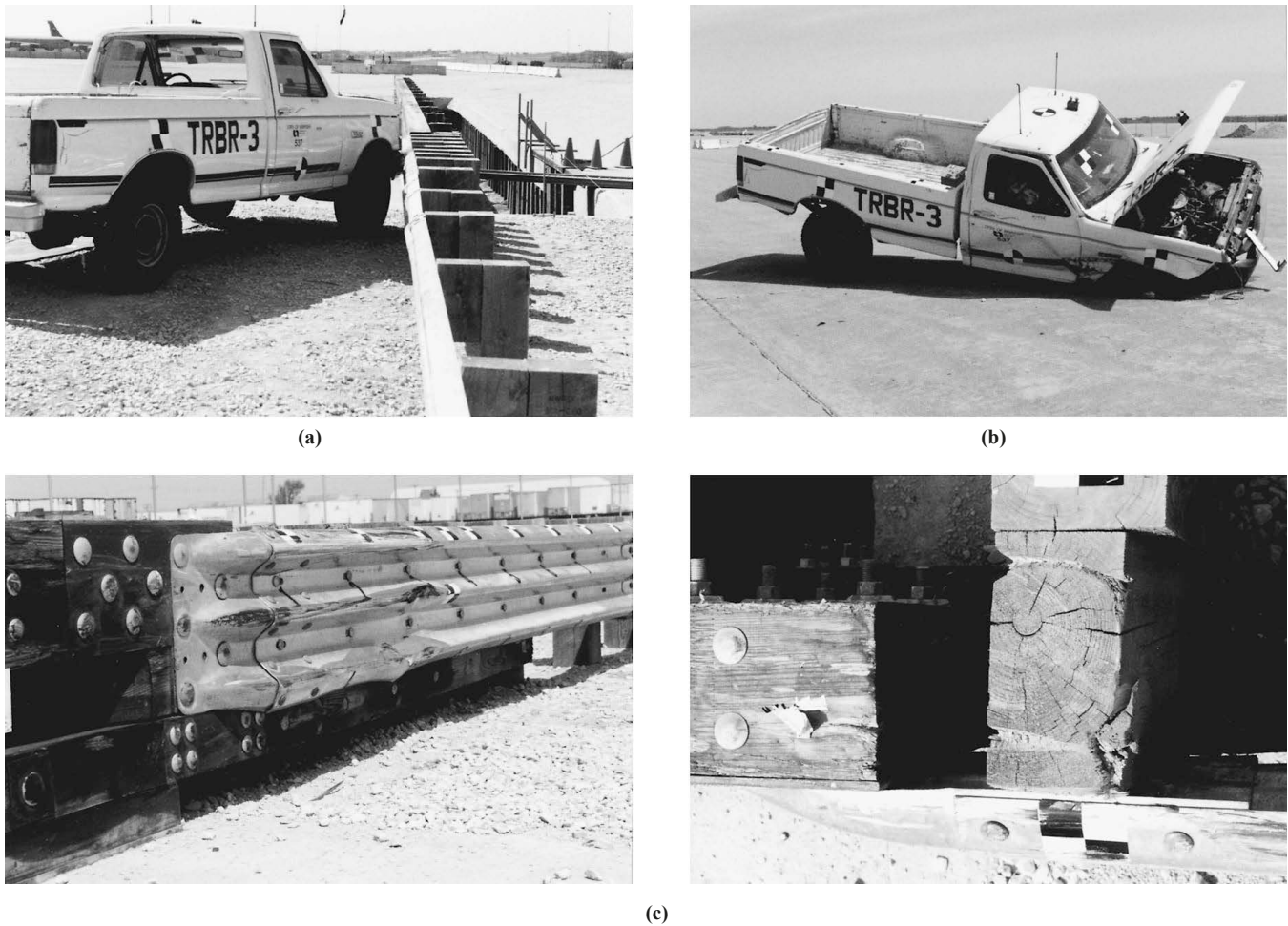


FIGURE 7 Test TRBR-3: (a) impact location, (b) vehicle damage, and (c) approach guardrail transition damage.

formed well as designed and that no design changes were necessary. For the steel system, the test results revealed the loads that imparted into key structural hardware were less than expected. For the two ASTM A325 bolts that measured 25 mm in diameter and that connected the post to the top mounting plate, the combined design load for both bolts was about 540 kN. However, the maximum combined bolt force was measured to be only about 470 kN. With this reduced loading into the plate assembly, the measured strain values near the outer regions of the top mounting plate were found to be about 10 to 12 percent of the values near the central region. Therefore, the researchers determined that the ASTM A307 bolts that measured 22 mm in diameter and that connected the top and bottom mounting plates to the deck should be reduced from 12 to 10.

CONCLUSIONS

This program clearly demonstrates that crash-worthy railing systems are feasible for transverse wood deck bridges. Even at high-impact conditions, such as those required by the TL-4 guidelines presented in *NCHRP Report 350*, the railing systems performed well, with no significant damage to the bridge superstructure. With

the development of crash-worthy railing systems, a significant barrier to the use of transverse wood deck bridges has been overcome. At the onset of this research program, no TL-4 crash-tested bridge railing systems were available for use on transverse wood deck bridges. Now, bridge engineers have two railing systems that are used on transversely laminated timber deck bridges located on higher-service-level roadways. Finally, an approach guardrail transition system has been developed and crash tested for use with each bridge railing system.

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Attachment 3



(a)



(b)



(c)

FIGURE 8 Test TRBR-4: (a) impact location, (b) vehicle damage, and (c) approach guardrail transition damage.

Attachment 3

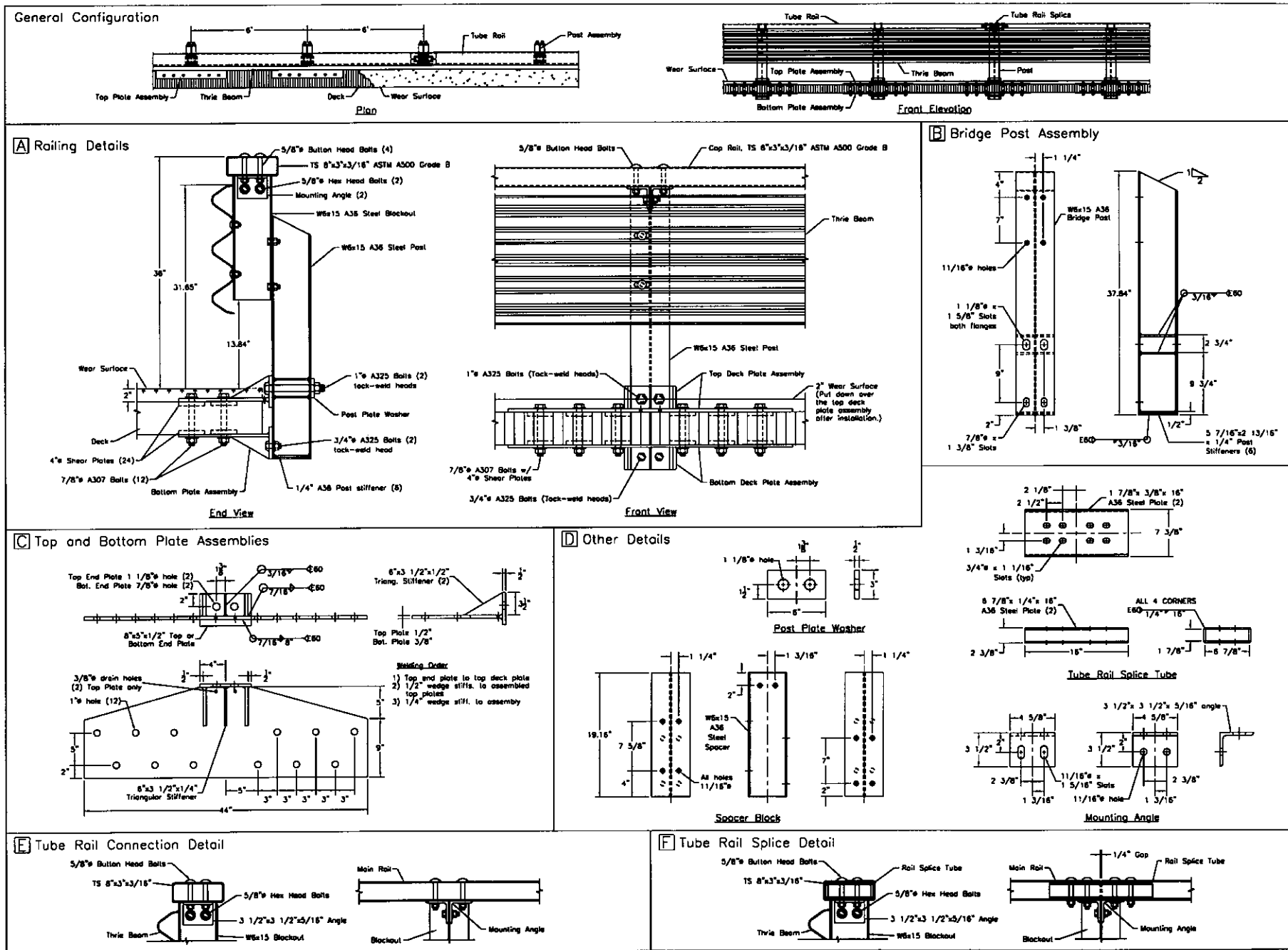


FIGURE 9 System No. 2: steel bridge railing design details (1 in. = 25.4 mm).

Attachment 3

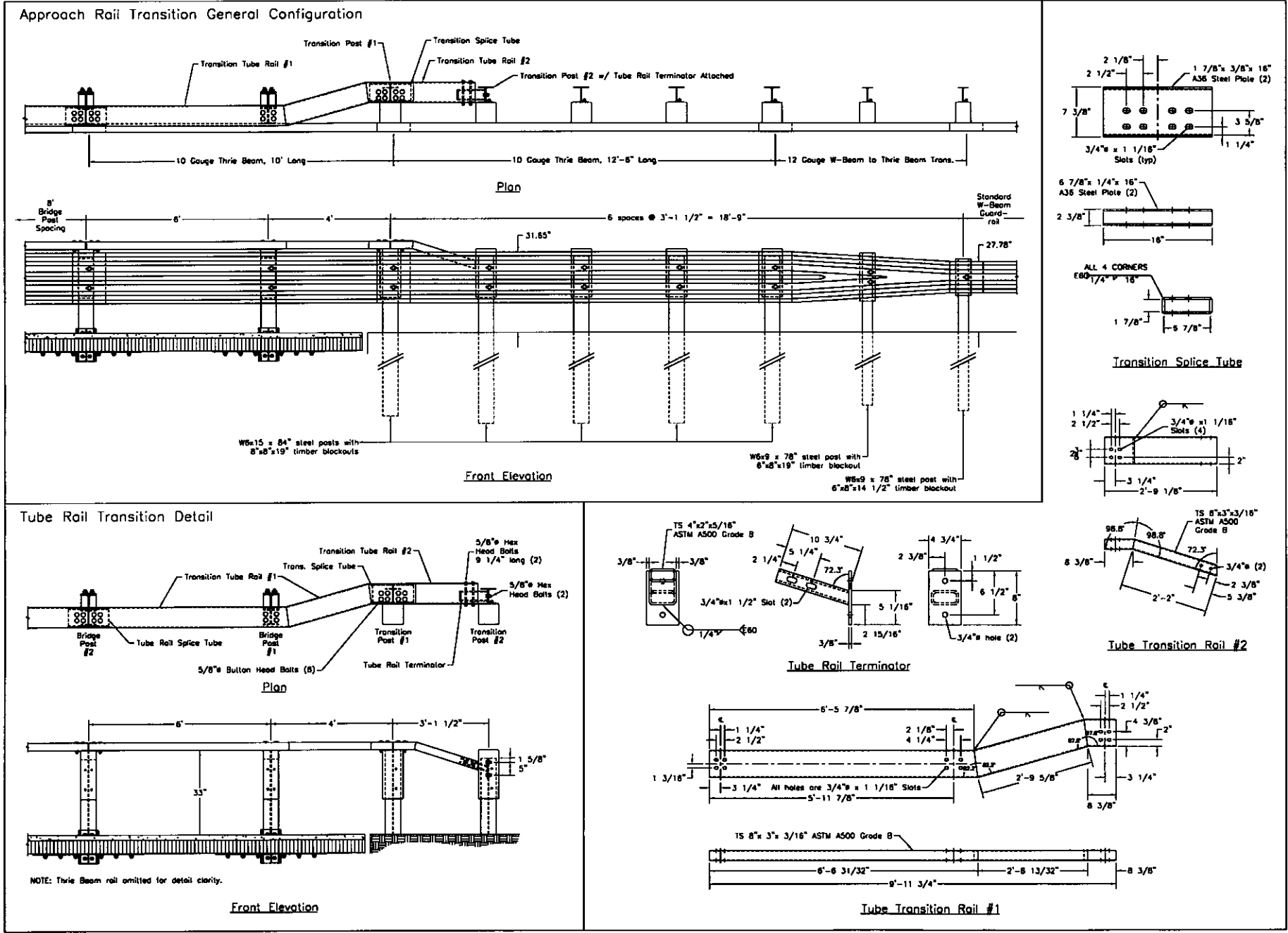


FIGURE 10 System No. 2: approach guardrail transition design details (1 in. = 25.4 mm).



(a)



(b)



(c)

FIGURE 11 Test STTR-1: (a) impact location, (b) vehicle damage, and (c) bridge railing damage.

Attachment 3



(a)



(b)



(c)

FIGURE 12 Test STTR-2: (a) impact location, (b) vehicle damage, and (c) bridge railing damage.

Attachment 3



(a)



(b)



(c)

FIGURE 13 Test STTR-3: (a) impact location, (b) vehicle damage, and (c) approach guardrail transition damage.



(a)



(b)



(c)

FIGURE 14 Test STTR-4: (a) impact location, (b) vehicle damage, and (c) approach guardrail transition damage.

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"Ronald K. Faller"
<rfaller1@unl.edu>

02/05/2013 11:36 AM

Please respond to
<rfaller1@unl.edu>

To 'Vue Her' <vue.her@dot.ca.gov>

cc 'Bob Meline' <bob.meline@dot.ca.gov>, "Ritter, Michael A
-FS" <mritter@fs.fed.us>, "Wacker, James P -FS"
<jwacker@fs.fed.us>, <rfaller@unl.edu>

bcc

Subject Bridge Railing Systems

Vue:

Over the years, MwRSF and the USDA-FPL-FS developed and crash tested 13 bridge railing systems for use on timber deck bridges. Two early railings systems were developed for 10.75-in. thick longitudinal glulam timber decks – a timber railing and curb system and a steel thrie beam and upper channel system (TRB 4th International Bridge Engineering Conference in Proceedings No. 7 and TRR No. 1500). These higher performance level systems were later adapted over to concrete decks, where bridge engineers would be responsible for ensure their retention to the deck with given design loads. Forest Product Laboratory design sheets were also prepared.

Later, these two bridge railings were modified to adapt them to 5.125-in. thick transverse glulam timber decks and evaluated under NCHRP 350 TL-4. I believe you have noted the latter two systems, which were published in TRR No. 1696.

Research reports are available for these railing systems. I also believe that the latter two system could be adapted over to concrete decks with consideration of edge deck design and adequate reinforcement. Look at it this way, the latter two systems worked on 5.125-in. thick wood decks.

All of the railing systems above are non-proprietary and available for use or modification by others! They exist for the taking. Let me know if you need anything else. Thanks for your interest.

P.S. – On a side note, MwRSF later modified 2 of the 13 railing systems for use on transverse nail-laminated decks for the State of West Virginia.

Ron

Ronald K. Faller, Ph.D., P.E.
Interim Director and Research Assistant Professor

Midwest Roadside Safety Facility (MwRSF)
Nebraska Transportation Center
University of Nebraska-Lincoln
130 Whittier Research Center
2200 Vine Street
Lincoln, Nebraska 68583-0853

(402) 472-6864 (phone)

(402) 472-2022 (fax)

rfaller1@unl.edu

From: Vue Her [mailto:vue.her@dot.ca.gov]
Sent: Tuesday, February 05, 2013 12:41 PM
To: rfaller1@unl.edu
Cc: Bob Meline
Subject:

Dear Dr. Faller,

We are looking for test level 4 side-mounted bridge rails and came upon FHWA's letter B-138. We notice that the letter only stated "for use on transverse, glue-laminated timber bridge decks." The middle paragraph mentions two bridge railings that meet TL-4 criteria, a wood laminated bridge rail and a steel bridge rail. I couldn't find any other information on these bridge rails. Are there limitations on installing these on concrete bridge decks and are these a proprietary?

http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road hardware/barriers/pdf/b138.pdf

Thank You,

Vue Her, M.S., P.E.
Office of Safety Innovation & Cooperative Research
Roadside Safety Research Group
5900 Folsom Blvd. MS-5
Sacramento, CA 95819-4612
(916) 227-5828