Development and Testing of Timber Bridge and Transition Rails for Transverse Glued-Laminated Bridge Decks

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Timber bridge and transition rails for transverse glued-laminated bridge decks were developed and tested. Three timber bridge rails with approach rails on both ends of the timber rail were developed for Performance Level 1, the lowest performance level. Two bridge rails did not have curb rails and one bridge rail had a curb rail. Six crash tests on three bridge rails and one crash test on a transition rail were performed. Each bridge rail was tested with an 817-kg (1,800-lb) small automobile and a 2452-kg (5,400-lb) pickup truck. The transition rail was tested with a 2452-kg (5,400-lb) pickup truck. All of the bridge rails and the transition rail met the crash test requirements as specified by the 1989 AASHTO Guide Specification for Rails and a 1981 NCHRP report. The crash test reports have been submitted to FHWA for review and acceptance.

Recent advances in timber bridge research and development have shown that timber can be a cost-effective and competitive bridge construction material. Timber bridges are being used for low to medium volume and high-intensity traffic conditions and share about 12 percent of total bridges that span 6.1 m (20 ft) or more in the United States (*1*). The present requirement for rail systems to be accepted by FHWA is that the rail system should be satisfactorily crash tested. The acceptance standards are given in AASHTO's 1989 Guide Specifications for Bridge Rails (*2*) and an NCHRP report (*3*). The lack of standard timber bridge rail systems that have been successfully crash tested in accordance with these specifications has limited the use of timber in bridge construction.

The Constructed Facilities Center of West Virginia University (CFC-WVU), was awarded a contract by FHWA to conduct timber bridge research. One task was to develop and crash test timber bridge rails and transition rails suitable for use on transverse timber bridge decks. Other research centers such as SouthWest Research Institute (SWRI) and Midwest Roadside Safety Facility are also involved in similar projects in developing and crash testing timber bridge rails according to the criteria specified in the AASHTO guide specifications for Performance Levels 1 and 2 (PL-1 and PL-2). To date, only one timber bridge rail, tested by SWRI, in 1988, is included in FHWA's approved list of bridge rails for federal-aid projects (*4*). This bridge rail was attached to a longitudinal dowel-laminated bridge deck and successfully crash tested for PL-1 criteria. PL-1 is specified for low-level bridges with light traffic.

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BRIDGE RAIL SYSTEMS

Bridge rails are provided to protect vehicle occupants and the traffic. Thus, bridge rails are important elements from a safety point of view. Approach rails are provided on both ends of the bridge rail and consist of both transition rail and guard rail. The transition rail connects the flexible guard rail to the rigid bridge rail.

Bridge rails are commonly made of concrete, steel, aluminum, or timber. The cross section of a bridge rail is a solid wall, a post-beam rail, or a combination of the two. From the functional point of view, bridge rails are classified as traffic rails, pedestrian rails, bicycle rails, and combination rails (*5*).

The main purpose of the traffic rails is to provide safety for the traffic by containing and redirecting the vehicle within the bridge. This is achieved by meeting geometric and strength requirements of the rails for crash testing. The following are important considerations to be taken into account in the design and evaluation of bridge and transition rails:

- Protection of vehicle occupants during collision with traffic rails,
- Protection of vehicle occupants and vehicles on the roadway and near the roadway,
- Replaceability,
- Aesthetics, and
- Cost minimization.

REQUIRED CRITERIA FOR EVALUATING BRIDGE RAIL CRASH TEST

The weight of the test vehicle (W), out-to-out wheel spacing (B), test vehicle center of gravity (CG) above the deck (G), impact angle (θ), and test vehicle velocity (V) are indicated in Table 1 for PL-1 according to AASHTO's guide specifications (*2*). The following are the required criteria for evaluating the bridge rail crash test according to these specifications:

1. The traffic bridge rail must contain the test vehicle without any penetration or without going over the test rail.
2. Debris penetration into the vehicle passenger compartment and hazards to other traffic caused by the crash test vehicle are not permitted.
3. The passenger compartment must show integrity without any deformation or intrusion.
4. The test vehicle must remain upright during and after impact.

TABLE 1 Crash Test Criteria of Bridge and Transition Rail for PL-1 (*2,3*)

Test Vehicle Description	Bridge Rail		Transition Rail
	Small Automobiles	Pick-up Truck	Pick-up Truck
Weight	817 kg (1.8 kips)	2452 kg (5.4 kips)	2452 kg (5.4 kips)
Out to Out Wheel Spacing	1.68 m (5.5 ft)	1.98 m (6.5 ft)	1.98 m (6.5 ft)
CG of the Vehicle Above the Deck	508 mm (20 in)	686 mm (27 in)	686 mm (27 in)
Distance from CG to Front of the Vehicle	1.65 m (5.4 ft)	(2.59 m) 8.5 ft	2.59 m (8.5 ft)
Impact Angle	20°	20°	20°
Performance Level	TEST VEHICLE SPEEDS		TEST VEHICLE SPEED
PL-1	80 kmph (50 mph)	72 kmph (45 mph)	72 kmph (45 mph)
Note: Permissible Variation in Test Vehicle Speed is +4/-1.6 kmph (+2.5/-1.0 mph) and Impact Angle is +2.5/-1.0 Degree.			

5. Occupant longitudinal and lateral impact velocities must be less than 9.2 and 7.6 mi/sec (30 and 25 ft/sec).
6. Occupant ride down longitudinal and lateral accelerations must be less than 15 *g*.

DESIRED CRITERIA FOR EVALUATING BRIDGE RAIL CRASH TEST

The following are the desired criteria for evaluating the bridge rail crash test according to AASHTO (*2*):

1. The test vehicle shall be redirected smoothly from the test article.
2. The rear of the test vehicle shall not yaw more than 5 degrees away from the rail during the impact and vehicle exit from the rail.
3. The effective coefficient of friction μ shall be less than 0.35. The smoothness of the rail is assessed by the effective coefficient of friction.
4. The test vehicle exit angle shall be less than 12 degrees.
5. The test vehicle shall not move more than 6.1 m (20 ft) laterally after the bridge rail impact. In addition, the maximum allowable lateral movement of 6.1 m (20 ft) should be maintained within the longitudinal vehicle travel, which is limited to 30.5 m (100 ft) plus vehicle length from the point of impact.

REQUIRED CRITERIA FOR RAIL DESIGN

The height of the rail from the top of the wearing surface to the top of a bridge rail shall be at least 686 mm (27 in.). An overlay thickness of 51 mm (2 in.) needs to be considered for the total height of the rail. Thus for the PL-1 rail system, the total height of the traffic rail shall not be less than 737 mm (29 in.) from the top surface of the deck.

The post setback distance (S) and the maximum clear opening below the bottom rail (C_b) and between the rails shall be determined

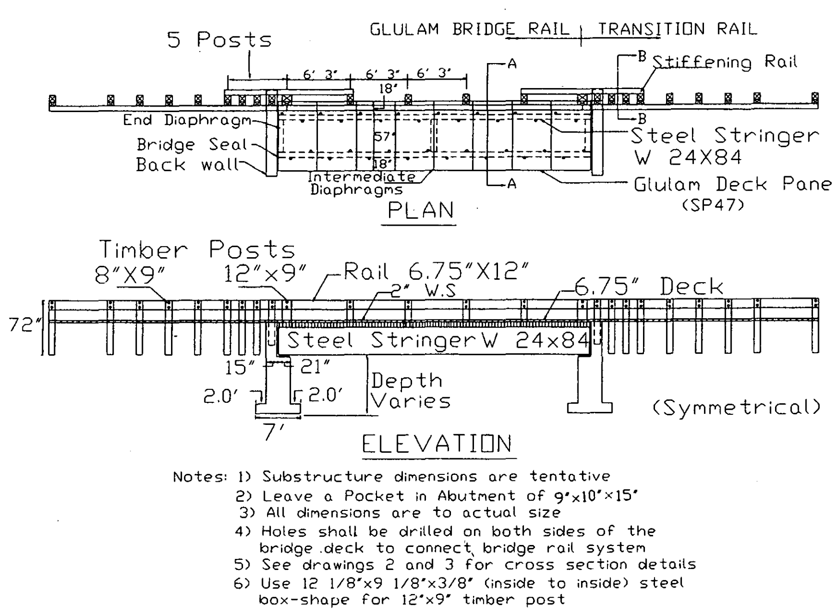

FIGURE 1 Timber Bridge Rail System 1 for PL-1: plan and elevation.

from Figure A 13.1.1-2 of Load Resistance Factor Design (LRFD)draft specifications (*5*). The bottom clear opening (C_b) shall not be greater than 381 mm (15 in.). The post setback distance shall not be less than 25.4 mm (10 in.) to avoid snagging of vehicle ports such as bumpers, wheels, and hood. The traffic face of rails must be continuous and smooth.

DEVELOPMENT OF TIMBER BRIDGE RAIL SYSTEMS FOR TRANSVERSE GLUED-LAMINATED BRIDGE DECKS

The timber bridge rail systems for transverse glued-laminated bridge decks were designed by CFC-WVU to meet the crash test requirements of PL-1. The crash tests were conducted at the Texas Transportation Institute (TTI). Three timber bridge rail systems are indicated in Figures 1 through 9 and listed as follows:

- Bridge Rail System 1: Glued-laminated rail attached to transverse timber deck on steel stringers,
- Bridge Rail System 2: Glued-laminated rail attached to transverse timber deck on glued-laminated beams, and
- Bridge Rail System 3: W-beam rail attached to transverse timber deck on steel stringers.

The following considerations were given while developing and selecting the feasible timber bridge rail systems:

- Ability to meet the strength requirements of AASHTO's 1989 guide specifications, (*2*) and NCHRP Report 230 (*3*),
- Replaceability of rail elements in the event of damage caused by vehicle impact,
- Adaptability to other types of timber bridge decks,
- Availability of material, including hardware,
- Maintainability and constructability,
- Structural integrity with the decks, and
- Cost and aesthetics.

GENERAL FEATURES OF BRIDGE RAIL SYSTEMS

The general features of each system along with the test criteria are indicated in Table 2. The bridge rails span 10 m (33 ft), with approach rails on both ends of the test bridge 7.6 m (25 ft) long. The height of the bridge rail from the top of the 51-mm (2-in.) wearing surface is 686 mm (27 in.) for all systems. The approach rail consists of a transition and a guard rail. All three bridge rail systems were developed for transverse glued-laminated timber bridge decks supported by glued-laminated and steel beams.

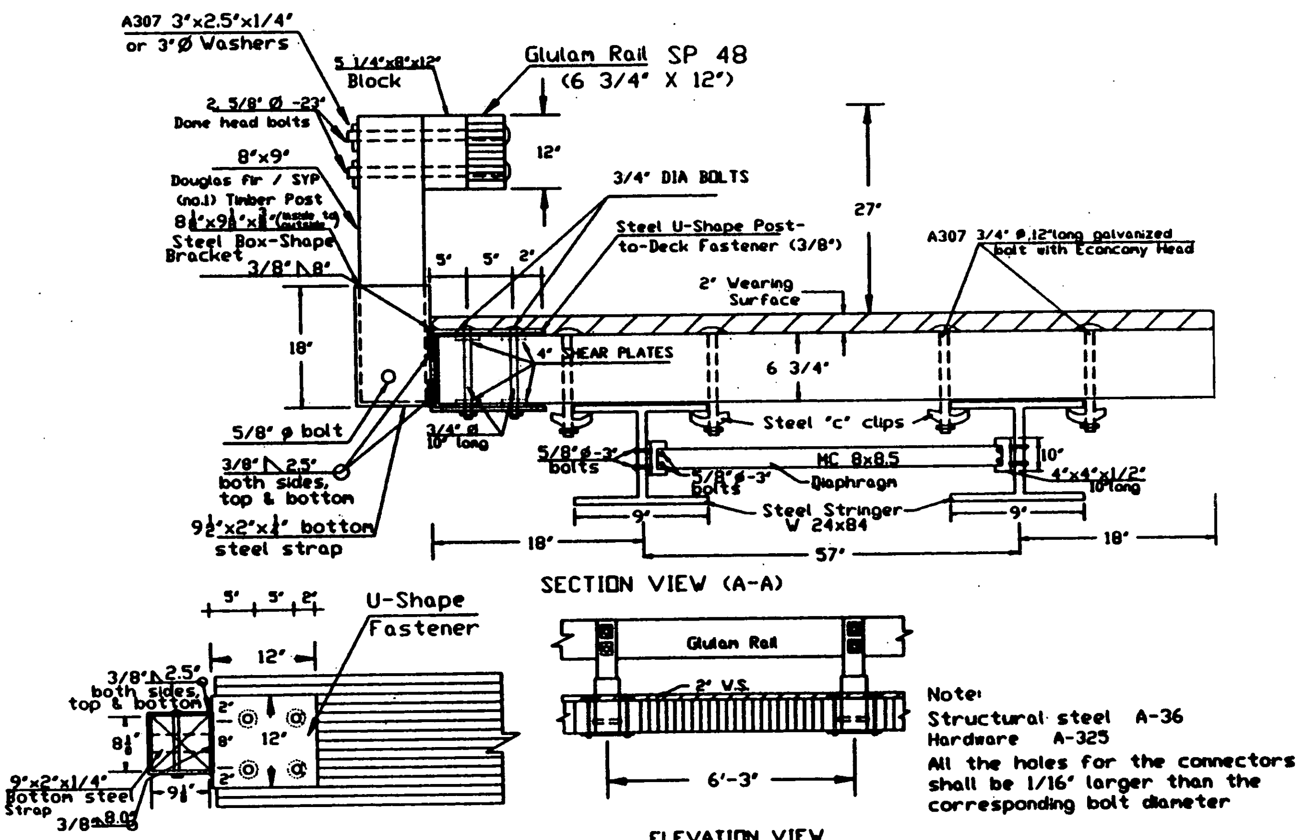

FIGURE 2 Timber Bridge Rail System 1 for PL-1: cross-sectional details of bridge rail.

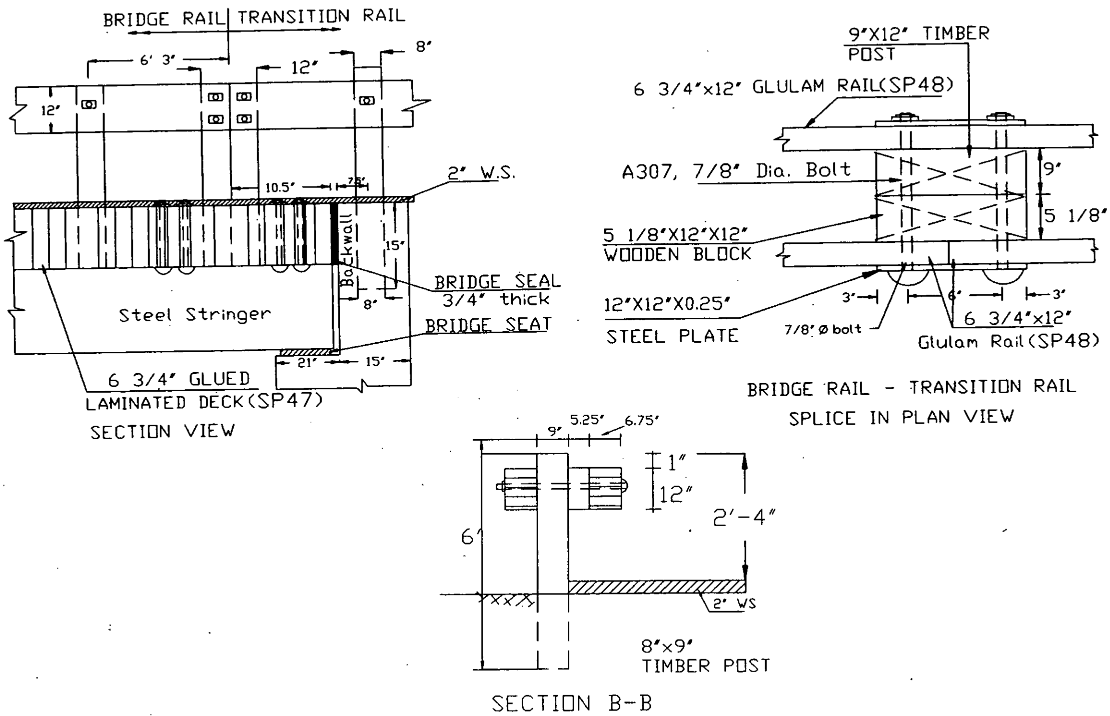

FIGURE 3 Timber Bridge Rail System 1 for PL-1: details of transition rail.

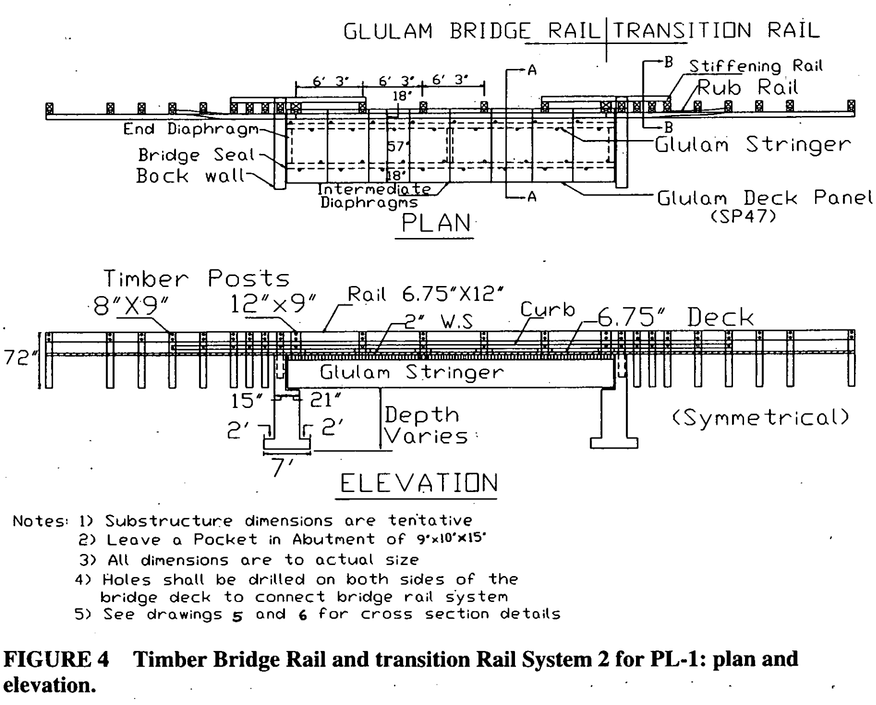

FIGURE 4 Timber Bridge Rail and transition Rail System 2 for PL-1: plan and elevation.

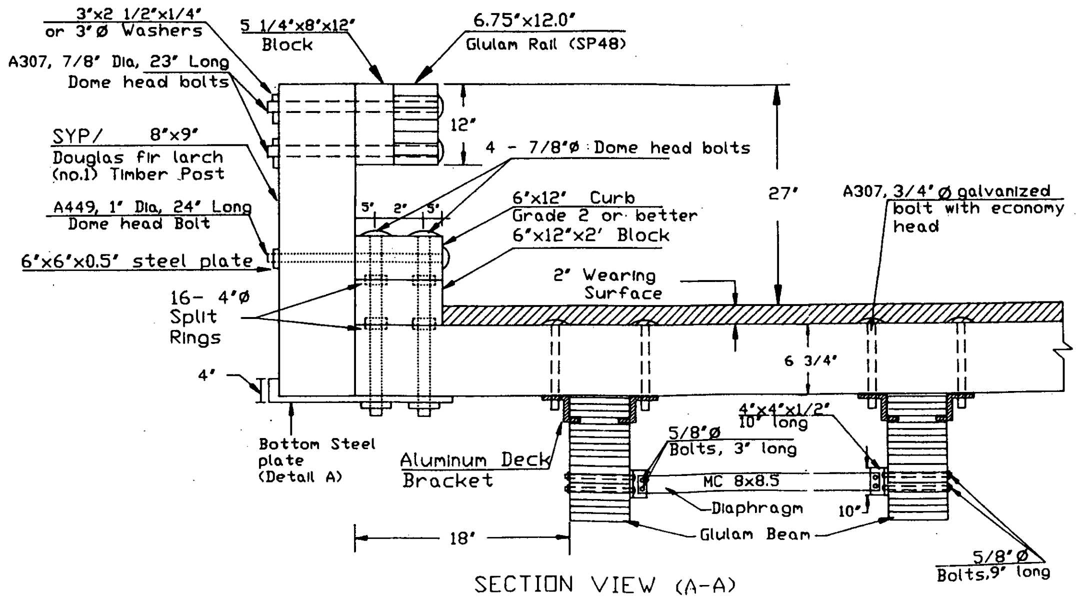

FIGURE 5 Timber Bridge Rail and transition Rail System 2 for PL-1: cross-sectional details of bridge rail.

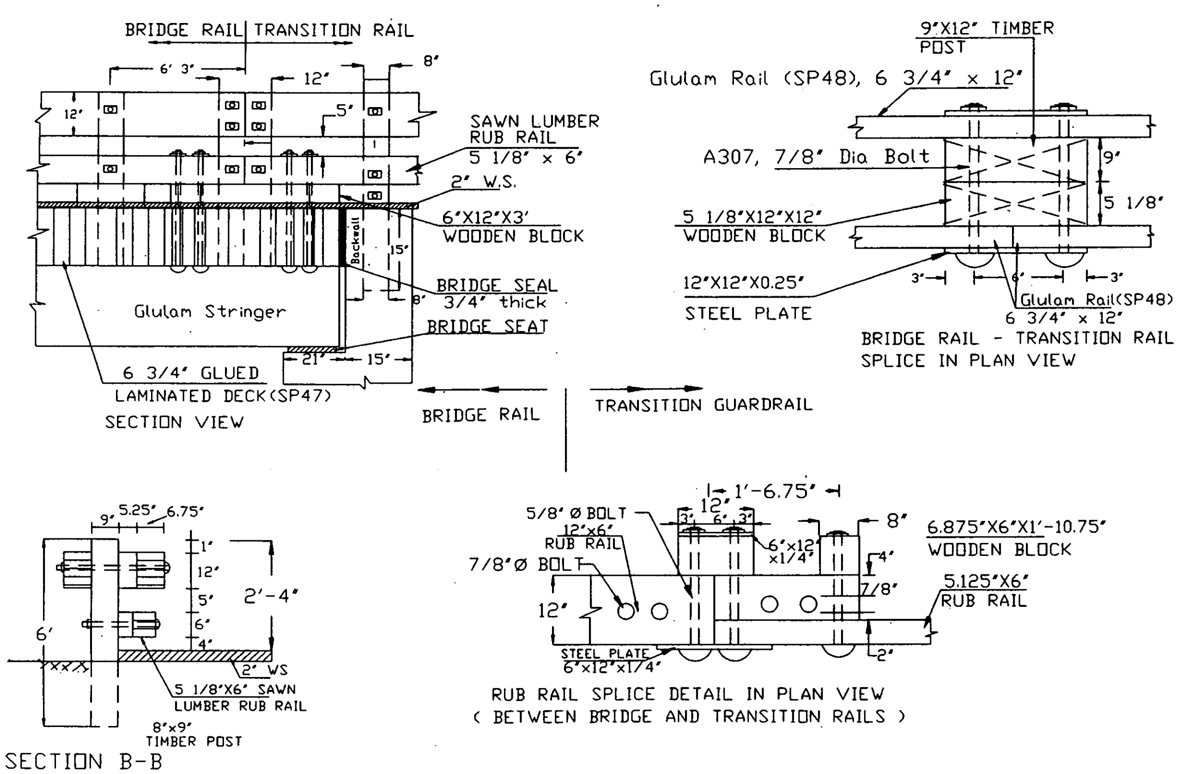

FIGURE 6 Timber Bridge Rail and transition Rail System 2 for PL-1: cross-sectional details of transition rail.

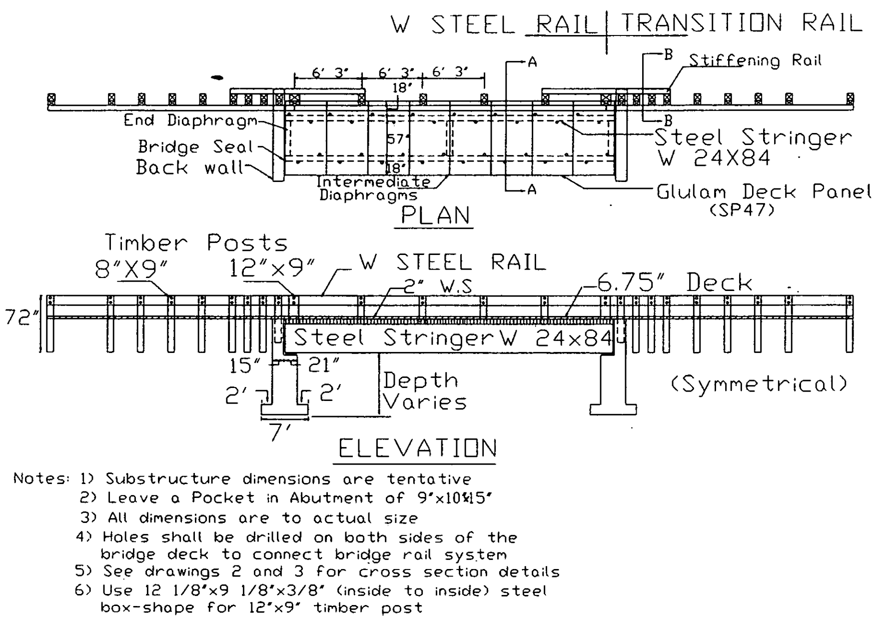

FIGURE 7 Timber Bridge Rail System 3 for PL-1: plan and elevation.

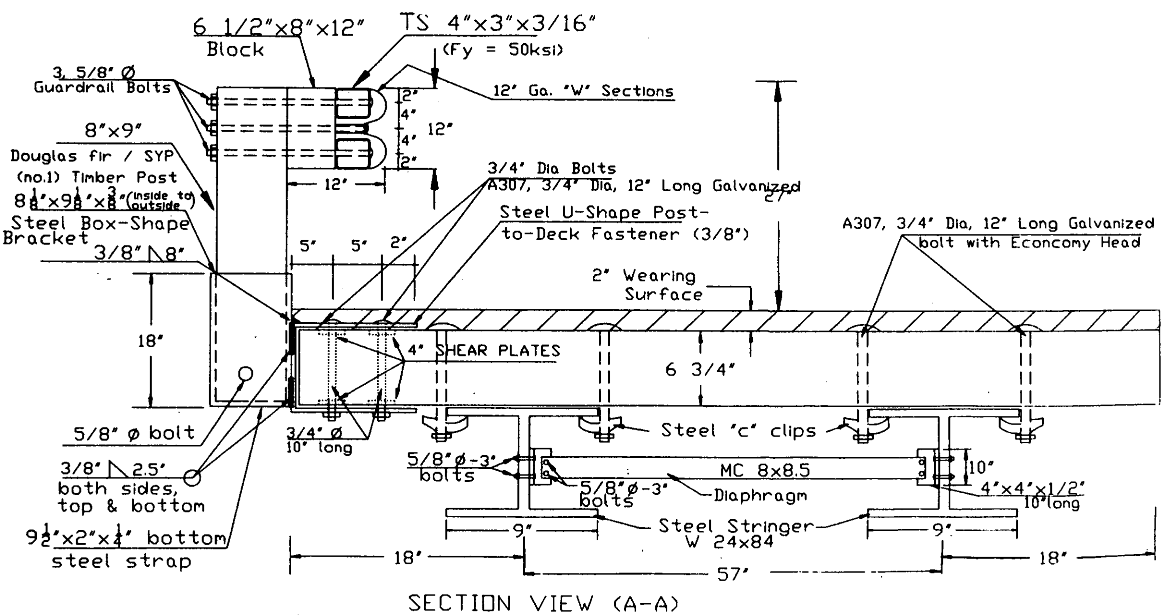

FIGURE 8 Timber Bridge Rail System for PL-1: cross-sectional details of bridge rail.

The deck was fabricated with eight glued-laminated panels; dimensions of each panel are 2.36 m (7 ft 9 in.) long, 1.26 m (4 ft 1.5 in.) wide, and 172 mm (6.75 in.) in depth. The bridge deck panels were connected to the supporting steel beams with "c" clips and to the glued-laminated beams with aluminum brackets. Systems 1 and 3 used two W 24 × 84 steel stringers, and System 2 used two glued-laminated beams 172 × 1029 mm (6.75 × 40.5 in.) to support the transverse glued-laminated bridge deck.

For the bridge rails, solid sawn lumber timber posts [203 × 229 × 927 mm (8 × 9 × 36.5 in.)] were used. These were spaced at 1.91 m (6 ft 3 in.) in all three systems. The post setback distance provided was 305 mm (12 in.) in all three systems. The same post size with varied post spacing was used in the approach rail. The first post in the transition rail closest to the bridge rail post was placed in the abutment, and the rest of the posts in the transition rail were embedded in the soil and compacted thoroughly.

Glued-laminated beam, 171 × 305 mm (6.75 × 12 in.), was used as the bridge rail and the approach rail for both Systems 1 and 2. W-beam mounted on two steel tubes [76 × 102 × 9.5 mm (3 × 4 × 3/8 in.)] was used as the bridge rail and approach rail for System

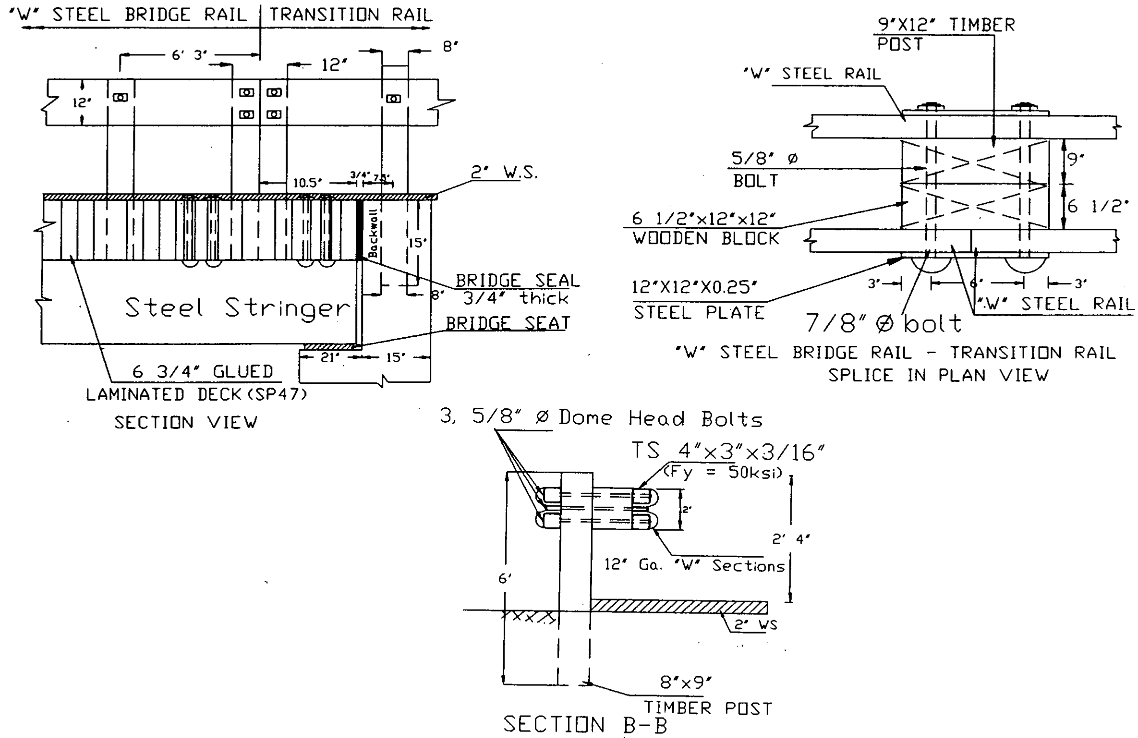

FIGURE 9 Timber Bridge Rail System 3 for PL-1: cross-sectional details of transition rail.

3. Systems 1 and 3 are identical, except that System 1 has a glued-laminated rail, whereas System 3 has a W-beam steel rail. Both Systems 1 and 3 do not have curb rails. The rails, attached to the wooden posts, are placed in box steel brackets. The box steel brackets are welded to steel U-shaped fasteners, as indicated in Figure 2. The steel U-shaped fastener is then connected to glued-laminated deck by four A325 dome head bolts 19 mm (3/4 in.) in diameter.

The bridge rail in System 2 has a curb rail. The posts are attached to the curb with bolts 25.4 mm (1 in.) in diameter. The curb is attached to the bridge deck with four dome head bolts 22 mm (7/8 in.) in diameter at each post location. The glued-laminated bridge rail is attached to the posts at the top with A307 bolts.

AASHTO guide specifications recommend an elastic analysis for the design of timber post, rail, deck, and structural connections

TABLE 2 Full-Scale Crash Test Based on AASHTO Load Criteria

Crash Test	Criteria	Test Criteria				Type of Test	Deck Supporting System	Type of Rail
		Type of Vehicle	Weight of Vehicle	Speed	Angle			
1	PL-1 System 1	Small Automobile	817 kg (1.8 kips)	80 kmph (50 mph)	20	Bridge Rail	WF Steel Beam	Timber
2		Pick-up Truck	2452 kg (5.4 kips)	72 kmph (45 mph)	20			
3	PL-1 System 2	Pick-up Truck	2452 kg (5.4 kips)	72 kmph (45 mph)	20	Transition Rail [1]	Glulam Beam	Timber
4		Small Automobile	817 kg (1.8 kips)	80 kmph (50 mph)	20	Bridge Rail		Timber
5		Pick-up Truck	2452 kg (5.4 kips)	72 kmph (45 mph)	20			
6	PL-1 System 3	Small Automobile	817 kg (1.8 kips)	80 kmph (50 mph)	20	Bridge Rail	WF Steel Beam	Steel
7		Pick-up Truck	2452 kg (5.4 kips)	72 kmph (45 mph)	20			

Notes: [1] This transition is compatible with rails in tests 1, 2, 4 and 5.

TABLE 3 Design Forces for Traffic Rails LRFD Recommendations (5)

Design Forces	PL-1
Lateral Force	12258 kg (27 kips)
Longitudinal Force	4086 kg (9 kips)
Vertical (Downward) Force	2043 kg (4.5)
Contact length for Lateral (L_t) and Longitudinal (L_L) forces	1.37 m (4.5 ft)
Contact length (L_v) for vertical forces	5.49 m (18 ft)
Height of Vehicle force (He)	508 mm (20 in)
Minimum height of rail (H) from top surface of the deck	737 mm (29 in)

TABLE 4 Design Values of PL-1 for Pickup Truck

Description	LRFD Draft Specifications	Design values considered by authors
Lateral Load	12258 kg (27 kips)	14528 kg (32 kips)
Contact length	1.37 m (4.5 ft)	1.07 m (3.5 ft)
Height of Rail	737 mm (29 in)	737 mm (29 in)
Recommended wearing surface	51 mm (2 in)	51 mm (2 in)
Vertical opening from bottom of the rail to top wearing surface	381 mm (15 in)	381 mm (15 in)
Height of vehicle force	508 mm (20 in)	584 mm (23 in)

under failure loads. Therefore, draft LRFD code recommendations for strength limit states and applicable load combinations were followed in the design of timber rails.

DESIGN OF BRIDGE RAIL SYSTEMS

The design of timber rails was performed by computing lateral force from a test vehicle using the following formulas according to AASHTO's guide specifications, (2):

$$F_{L.AV} = \frac{W V_I^2 \sin^2\theta}{2g[A \sin\theta - B(1 - \cos\theta) + D]} \qquad (1)$$

where
W = gross weight of the vehicle (lb),
V_I = impact velocity in (ft/sec),
θ = impact angle (degrees),
g = acceleration as a result of gravity (ft/sec^2),
A = distance from CG to front of the vehicle (ft),
B = outer-to-outer wheel spacing (ft), and
D = Barrier deflection.

$$F_{L.\max} = \frac{\Pi}{2} F_{L.AV} \qquad (2)$$

Lateral forces were computed using Equation 2, and these lateral forces have compared well (±20 percent) with the experimental results. The contact length of 89 mm (3.5 ft) was used to distribute the lateral load (6).

The allowable design stresses were arrived at by taking the recommended design stresses from the National Design Specification (NDS) (7). The recommended design stresses for bending, shear, and compression perpendicular to grain were adjusted with appropriate adjustment factors (Table 2.3.1, NDS)—impact factor and safety factor—as indicated in Equation 3:

$$F_A = C_A C_I C_s F_D \qquad (3)$$

F_A = allowable design stress,
F_D = recommended design stress according to NDS,
C_A = appropriate adjustment factors,
C_I = impact factor, and
C_s = safety factor.

The impact factor is taken as 1.65, according to AASHTO's 1989 specifications (2). The factor of safety for ultimate strength to allowable design strength is about four for bending, shear, and compressive stresses. However, in designing the timber bridge rails, the allowable design stresses were multiplied by the factor of safety of two. The actual factor of safety, which is greater than two, was not taken into account intentionally to keep a substantial reserve strength in the material in the event of excessive force inducement by crashing at higher speeds or at heavier vehicle loads.

The design forces considered in the analysis for designing the rail systems are indicated in Table 3. Additional design details recommended previously (5) in Section 13 for bridge rail systems are shown in Table 4. It can be seen from Table 4 that the author's design is conservative by about 15 percent because the lateral force considered by the authors is higher than that recommended in the LRFD bridge design specifications (5). The maximum lateral force from the pickup truck was estimated using Barrier VII (8) and found to be about 12 258 kg (27 kips), which is equal to the lateral force recommended in the LRFD specifications.

Glued-laminated and W-beam steel rails, timber posts, and glued-laminated deck and structural connections are designed for bending strength, shear strength, and bearing strength. The computed stresses for rail and post of all three systems and allowable design stresses are indicated in Table 5. The computed stresses are lower than the allowable design stresses, except for the shear stress in the post, which is about 50 percent higher than the allowable design stress. The high shear stress in the post is because the factor of safety in the allowable design shear stress to ultimate shear strength is about four. However, in the rail design, a factor of safety of two was used; thus there was twice the reserve strength left to resist the shear stress in the post.

After the construction drawings were prepared for all three systems, the timber bridges were fabricated by Burke, Parsons and Bowlby Corporation of West Virginia. All three systems were fabricated and assembled in about 2½ months. No fabrication and assembly problems were encountered during the construction of the systems. The assembled systems were then shipped to the TTI for

TABLE 5 Stress Levels of Various Components for Different Rail Systems

Description	Components	Stresses (kpa)					
		Bending		Shear		Bearing	
		Computed	Allowable	Computed	Allowable	Computed	Allowable
System 1	Rail (SP48)	24666	30316	2480	3445	2274	6201
	Post (SP)	27422	27216	3927	2067	31418	22392
System 2	Rail (SP 48)	24666	30316	2480	3445	2273	6201
	Post (SP)	27422	27216	3927	2067	6201	6201
System 3	Rail (W-BEAM STEEL)	358348	344500	35277	199810	3445	22396
	Post (SP)	27422	27216	3920	2577	31418	22396

Note: 1 psi = 6.89 kpa

TABLE 6 Evaluation of Crash Tests

Sl. #	Criteria	System One		System Two			System Three		Pass/ Fail
Required Criteria		Test 1 Small Car	Test 2 Pick-up Truck	Test 3 Pick-up Truck	Test 4 Small Car	Test 5 Pick-up Truck	Test 6 Small Car	Test 7 Pick-up Truck	
A	Must contain vehicle	Vehicle was contained in all tests							Passed
B	No debris penetration into passenger compartment	No debris was penetrated into passenger compartment in all tests							Passed
C	No deformation passenger compartment	No passenger compartment deformation was found in all tests							Passed
D	Vehicle must remain upright	Vehicle remained upright in all tests							Passed
E	Occupant longitudinal impact velocity < 9.2 m/sec	5.6	3.8	3.9	4.7	3.4	4.4	4.4	Passed
	Occupant lateral velocity < 7.6 m/sec	5.7	3.8	3.3	5.3	3.3	6.4	4.1	Passed
	Occupant ride down longitudinal acceleration < 15 g's	-1.9	- 8.6	-3.5	-1.0	-3.4	-1.3	-3.7	Passed
	Occupant ride down lateral acceleration < 15 g's	-2.6	-14.7	-6.3	-4.1	-9.0	-5.3	-8.8	Passed
DESIRED CRITERIA									
F	Effective Coefficient of friction μ Assessment 0-0.25 Good 0.25-0.35 Fair > 0.35 Marginal	0.62 Marginal	0.42 Marginal	0.31 Fair	0.32 Fair	0.14 Good	0.34 Fair	0.4 Marginal	Passed

crash testing. The assembled systems were easily erected at the TTI using two fork lifts. The bridge rails were then crash tested for PL-1 with a small automobile weighing 817 kg (1,800 lb) and a pickup truck weighing 2 452 kg (5,400 lb). The transition rail was crash tested with a pickup truck weighing 2 452 kg (5400 lb).

Test 1 with the small automobile and Test 2 with the pickup truck were performed at about midspan of the bridge rail on timber Rail System 1. Test 3 with the pickup truck was performed on the transition rail attached to System 2. Test 4 with a small automobile and Test 5 with a pickup truck were performed at about midspan of the bridge rail on timber rail System 2. Similarly, Test 6 with a small automobile and Test 7 with a pickup truck were performed at about midspan of the bridge rail on timber Rail System 3. All seven tests met the crash test criteria specified in AASHTO's 1989 specifications for rails (*2*) and NCHRP Report 230 (*3*). The bridge rails and the transition rail performed well by containing and redirecting the vehicle with cosmetic damage to the rails. The results of all seven tests are indicated in Table 6.

SUMMARY

1. Three timber bridge rail systems for transverse timber decks (one with a curb and two without curbs) were developed with approach rails on both ends of the bridge for PL-1 according to the requirements of AASHTO's 1989 guide specifications (*2*) and NCHRP Report 230 (*3*).
2. All three rail systems were well instrumented.
3. The test results of the seven tests were within the limits of the specified crash test criteria.
4. The design lateral forces recommended by the LRFD bridge design specifications (*5*) compared well with the experimental values, and the induced stresses were within the allowable limits.
5. The presence of curb rail improves the stiffness of the system and results in better performance of the system.
6. The reports will be submitted to FHWA for its review and acceptance into its list of approved bridge rails for federal-aid projects.

ACKNOWLEDGEMENTS

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DISCUSSION

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The authors address an important topic and attempt to develop and test three bridge railings for use on transverse glue-laminated timber bridge decks that could meet PL-1 criteria of AASHTO (*1*). The demand for crashworthy railing systems has become more evident with the increasing use of timber bridge decks on low-volume county and local road systems. Unfortunately, several flaws in the research approach seriously undermine the value of the bridge railings and approach transitions described in the foregoing paper. The following discussion is submitted with respect to several technical issues, such as rail length, impact location, discussion on timber deck damage, and discussion on approach transitions.

The authors reported the length of the bridge deck and rail to be approximately 33 ft. However, the approach transition incorporated a "stiffening rail" or backup rail that extended 7 ft 1½ in. onto both ends of the bridge rail, reducing the actual rail length to only 18 ft 9 in. A bridge rail length of 18 ft 9 in. is neither sufficient nor acceptable for crash testing bridge rails. Testing railings of insufficient length often artificially increases the rail's structural capacity, especially when strong transition and backup rails are incorporated. According to an NCHRP report (*2*), the recommended test length for a bridge rail, excluding terminals, should be at least three times the length in which deformation is predicted, but not less than 75 ft. AASHTO's guide specifications (*1*) follow the guidelines set forth by this NCHRP report (*2*). The new crash testing guidelines found in another NCHRP report (*3*) have similar recommendations for rigid bridge rails, but with the added stipulation that flexible bridge rails should not be less than 98 ft long. One purpose of these minimum bridge rail length recommendations is to ensure that full-scale vehicle crash tests are conducted beyond the strengthening effects of stiff transition designs such as the one incorporated in the foregoing paper.

The lack of sufficient bridge rail length is even more pronounced when it is considered in light of the impact point used for all six full-scale vehicle crash tests conducted on the three bridge rails. The authors used a midspan impact point, which meant that only 9 ft 4½ in. of unstiffened rail remained in front of the vehicle at the time it struck the bridge rail. Crash testing and computer simulation have indicated that the maximum lateral impact forces transmitted to barriers during large automobile and pickup truck impacts are applied 4 or 5 ft downstream from the point of impact (*3*). Thus, the maximum lateral impact forces were applied only 4 or 5 ft from the start of the stiffened transition section where the bridge rail's strength and stiffness are artificially elevated. Therefore, the short bridge rail length and impact location invalidates all crash test results. The reader should be cautioned against using these bridge rail designs on any bridge longer than 33 ft or without incorporating the stiff transition designs developed under this research effort.

The authors stated that only cosmetic damage occurred to the bridge rails and transition rail. No damage was reported to have occurred to the timber bridge deck panels (i.e., cracking of the timber deck panels). However, from the crash test reports for Systems 1 (*4*) and 2 (*5*), more significant damage in the form of permanent residual displacement of the timber deck was reported to have occurred. For the pickup truck crash test on System 1 (*4*), residual displacement was reported on four of the eight deck panels ranging from 0.5 to 0.75 in. For the minicompact and pickup truck crash tests on System 2 (*5*), the maximum residual displacement of the timber deck panels was 0.25 and 0.5 in., respectively. This bridge deck damage is much more significant than if only small cracks appeared on the surface of the timber deck. The amount of deck damage described in the crash test reports (*4,5*) would be associated with extremely high maintenance and repair costs. Such high maintenance and repair costs are an important consideration when selecting bridge rail systems, even for low-volume roads where accident frequencies are expected to be low.

The authors performed one additional full-scale vehicle crash test on an approach transition attached to System 2. However, no information was provided about the impact location and selection. In addition, design details provided for transitions attached to Systems 1 and 3 would lead the reader to believe that these systems have been successfully crash tested. However, no full-scale vehicle crash tests were performed on the transitions attached to Systems 1 and 3.

On the basis of the insufficient bridge rail length, inappropriate impact location, damage to the timber deck panels, and inappropriate testing or documentation of the transition designs, or both, bridge engineers and designers should use caution when specifying any of the bridge railing and approach transition systems described in the foregoing paper.

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AUTHORS' CLOSURE

The discussant's interest in this paper and the unpublished final reports on the crash test results are appreciated. A 33-ft span was selected for study on the basis of analysis and because most timber bridges are short-span bridges in the range of 22 to 44 ft in length. Developing and crash testing a 75-ft bridge rail were not deemed necessary because of timber's good energy-absorbing capability and because it would not represent a typical case were these rails ever to be involved in real accidents.

Each system that was tested included both the bridge rail and the transitions attached to it, which is how an actual system would be built. The transition that was tested in System 2 is designed for use with System 1 as well. All the rails tested performed to the Performance Level 1 criteria set forth in the AASHTO *Guide Specifications for Bridge Railings*. Each system contained and smoothly redirected the test vehicles. There were no debris or detached elements from the bridge rails that could potentially penetrate the occupant compartment. The vehicles remained upright and stable during the collision sequence.

The short length does lead to load sharing between the rail and the transitions. However, the posts that were instrumented show that 65 to 70 percent of the total impact force was taken by the nearest post. There was no pocketing of the rails, and the posts did not fail. The impact location was chosen to be between the posts to study the shear response of timber rails, because timber is weak in shear that is perpendicular to the grain. The rails performed well in the crash tests. If these rails are to be used in bridges with lengths much greater than 44 ft, a case could be made for retesting the bridge rail separately without attaching the transitions.

On deck displacement, ¼ to ½ in. of the deck displacement occurred between the permanent test pavement and the timber deck. It is believed this was because the deck was not rigidly anchored to the abutments. The deck in the test installation is only 8 ft wide as opposed to 20 ft or more in actual applications. The ratio of impact force to bridge deck weight is therefore substantially higher for the test installation. Thus, displacement of the bridge deck may not occur in actual applications.

In conclusion, the authors acknowledge that the length of the bridge rails was shorter than is customary for crash testing bridge rails. This was done to provide a more realistic test condition. The rails met the AASHTO recommendations in that they redirected the vehicle and provided minimal damage to the occupant, vehicle, and rail system. The authors believe the systems developed are adequate for timber bridges with transversely laminated decks of this approximate length.

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