Performance Level 1 Bridge Railings for Timber Decks

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Historically, very little research has been conducted for developing crashworthy railing systems for timber bridge decks. For timber to be a viable material in the new construction of bridges, vehicular railing systems must be developed and crash tested. The USDA Forest Service, Forest Products Laboratory, in conjunction with the Midwest Roadside Safety Facility, undertook the task of developing and testing three bridge railings—two glulam timber railing systems and one steel railing system—for use with longitudinal timber bridge decks. This research effort provided a variety of aesthetically pleasing and economical bridge-railing systems for timber decks. As part of the project, a series of four full-scale crash tests was conducted on the bridge-railing designs. The tests were conducted according to Performance Level 1 (PL1) specified in the AASHTO *Guide Specifications for Bridge Railings* (1989). The safety performance of each of the three bridge railings was acceptable according to the PL1 guide specifications.

Historically, most crashworthy bridge-railing systems located along highways have been developed using materials such as concrete, steel, and aluminum. In addition, most of these railing systems have been constructed on reinforced-concrete bridge decks. The demand for crashworthy railing systems has become more evident with the increasing use of timber bridge decks in secondary highways, county roads, and local road systems. Very little research has been conducted for developing crashworthy railings for timber bridge decks. For timber to be a viable material in the construction of bridges, additional vehicular railing systems must be developed and crash tested for timber bridge decks.

Of the many railing systems that have been crash tested successfully, only one involved a bridge railing for a timber bridge deck. This one study of significance was a 1988 Southwest Research Institute (SwRI) evaluation of a longitudinal glulam timber and sawn lumber curb system attached to a longitudinal spike-laminated timber deck (*1*). The evaluation was conducted according to Performance Level 1 (PL1) criteria specified in the AASHTO *Guide Specifications for Bridge Railings* (*2*).

Although the system met AASHTO PL1 requirements, damage consisting of delamination of the deck timbers and minor pull-out of several spikes was observed. As a result, the system has not been widely implemented in the field, and there continues to be a demand for crashworthy bridge-railing systems that would not cause damage to timber decks.

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OBJECTIVE

The objective of this research project was to develop bridge-railing systems for timber bridge decks while addressing concerns such as aesthetics and economy. The U.S. Department of Agriculture (USDA) Forest Service, Forest Products Laboratory, in cooperation with the Midwest Roadside Safety Facility (MwRSF), undertook the task of developing three bridge railings—two glulam timber bridge-railing systems and one steel bridge-railing system—that would be compatible with the existing types of longitudinal timber bridge decks. The longitudinal glulam timber bridge deck was selected because it was determined to be the weakest existing timber deck for transverse railing loads. If the bridge railings performed successfully on the glulam bridge deck, then the railing designs would be adaptable to most other longitudinal timber bridge-deck systems with no reduction in the railing performance depending on the specific railing systems. The bridge railings were developed to meet the AASHTO PL1 criteria while causing no damage to the longitudinal glulam timber deck.

The bridge railings included the following:

- A longitudinal glulam timber and sawn lumber curb bridge railing, the "curb system" (Figure 1),
- A single longitudinal glulam timber bridge railing without curb, the "shoe-box system" (Figure 2), and
- A single steel thrie-beam bridge railing, the "steel system" (Figure 3).

EVALUATION CRITERIA

The three bridge-railing systems were evaluated in accordance with the requirements for PL1 described in AASHTO (*2*). The full-scale crash tests were conducted and reported in accordance with *NCHRP Report 230* (*3*) as required by AASHTO (*2*). To be accepted for use in new construction, the bridge-railing systems were required to satisfy the performance evaluation criteria from two full-scale crash tests. The two required PL1 tests consist of a 2452-kg (5,400-lb) vehicle and an 817-kg (1,800-lb) vehicle impacting at 72.4 and 80.5 km/hr (45 and 50 mph), respectively, with impact angles of 20 degrees.

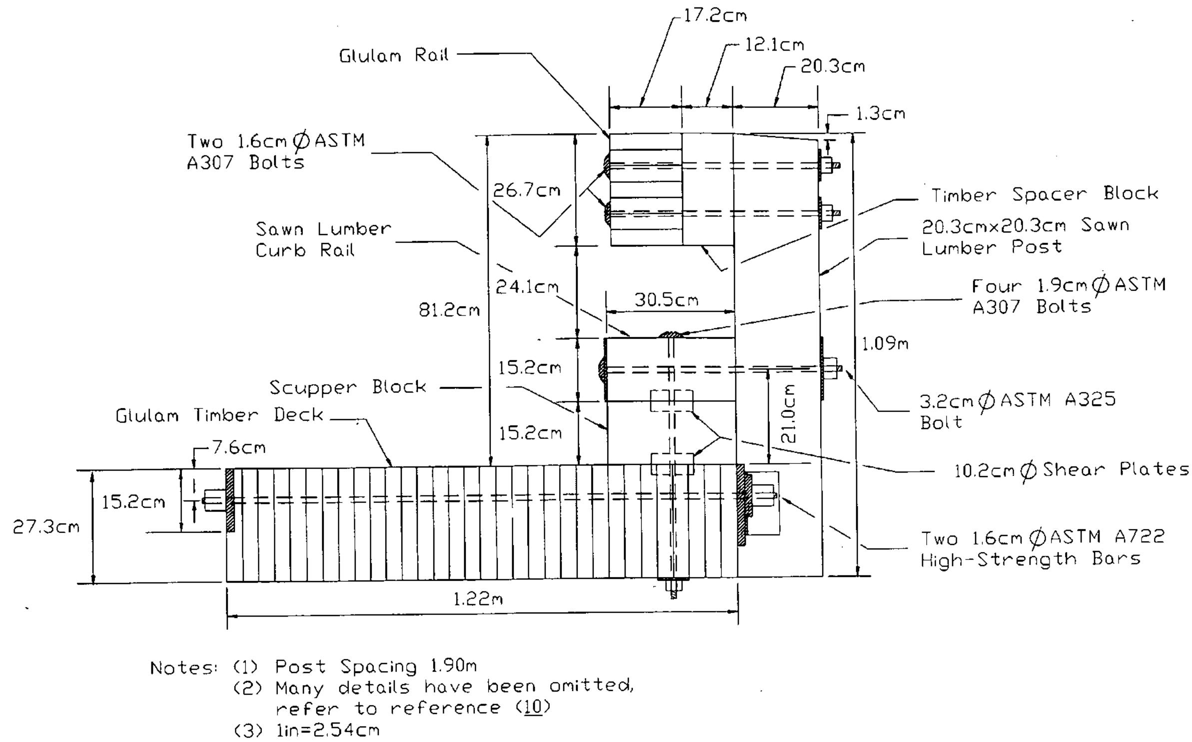


FIGURE 1 Longitudinal glulam and sawn lumber curb bridge railing, or curb system.

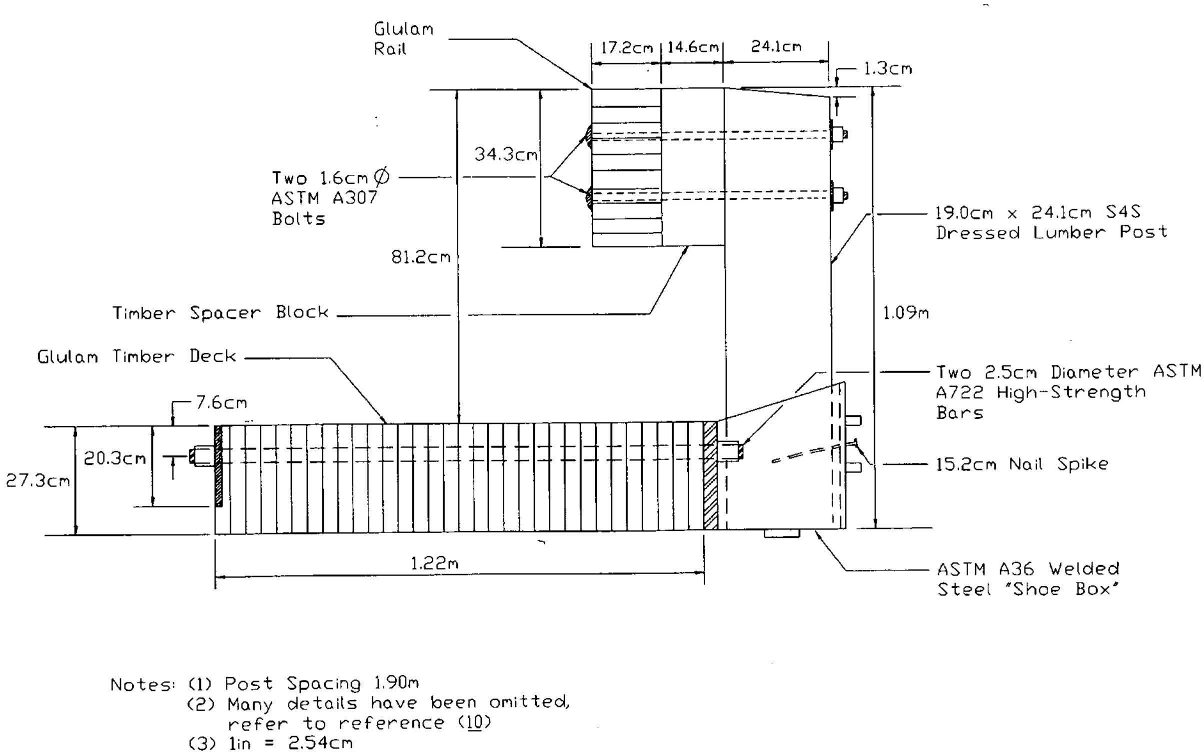


FIGURE 2 Single longitudinal glulam bridge railing, or shoe-box system.

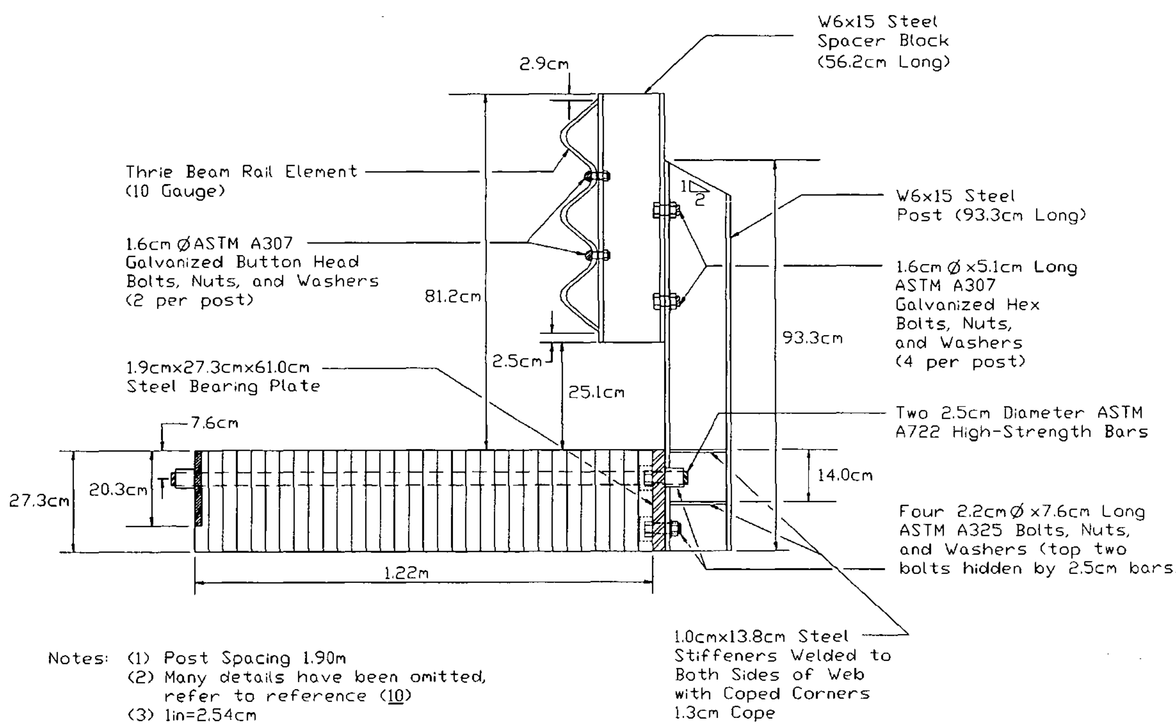


FIGURE 3 Single steel thrie-beam bridge railing, or steel system.

RESEARCH REVIEW AND APPROACH

In designing vehicular railing systems for highway bridges, engineers have traditionally assumed that vehicle impact forces can be approximated by applying equivalent static loads to the post and rail elements. Although rail loads are actually dynamic, the equivalent static load method has been used for years as a simplified approach to standardized railing design. The AASHTO *Standard Specifications for Highway Bridges* (*4*) currently requires that rail posts be designed to resist an outward transverse static load of 44.5 kN (10 kips). A portion of this load is also applied to posts in the inward transverse, longitudinal, and vertical directions and to the rail elements. These requirements are identical for all bridges irrespective of bridge geometry or traffic conditions; for example, railings for a single-lane bridge on a low-volume road must meet the same loading requirements as a bridge on a major highway. Computer simulation modeling and full-scale crash testing, however, have demonstrated that this procedure does not accurately estimate the actual vehicular impact forces transmitted to a bridge railing.

Specific requirements for bridge-rail crash testing have generally followed *NCHRP Report 230* procedures (*3*). In 1989 AASHTO adopted the *Guide Specifications for Bridge Railings* (*2*), which outlined the crash test requirements for three performance levels (PL1, PL2, and PL3); these specifications recommend full-scale vehicle crash testing of all railings used in new construction projects. In recent years, FHWA has recommended that full-scale crash testing be completed in accordance with the AASHTO guide specifications (*2*) for all vehicular railing systems. The testing requirements are based on the performance level of the bridge, which is defined by a number of factors including design speed, average daily traffic (ADT), percentage of trucks, bridge-rail offset, and number of lanes.

CURB SYSTEM

System Development

The timber bridge-railing system tested by SwRI provided the basis for the first design for the longitudinal glulam timber deck, although modifications and improvements were made to improve structural efficiency and reduce the cost of the system. Development of the modified longitudinal glulam timber and sawn lumber curb bridge railing, or curb system, consisted of redesigning the structural components and load transfer mechanisms used with the system previously tested on the spike-laminated timber deck.

The system evaluated at SwRI was constructed and tested with sawn lumber posts measuring 20.3 × 30.5 cm (8 × 12 in.) deep to accommodate AASHTO PL2 requirements. Researchers determined that this post size was in excess of that required for PL1, and that posts 20.3 × 20.3 cm (8 × 8 in.) would provide sufficient strength. The earlier system had also been constructed with a non-standard-size glulam rail of 15.2 × 27.3 cm (6 × 10¾ in.), whereas in the redesigned curb system, the rail section was modified to 17.1 × 26.7 cm (6¾ × 10½ in.) in order to obtain a more universal and economical size (*5*). The curb rail was maintained at a nominal size of 15.2 × 30.5 cm (6 × 12 in.). Hardware was also modified: in the previous system, the curb rail was attached to the deck with four ASTM A325 bolts 1.9 cm (¾ in.) in diameter, whereas for the curb system, it was assumed that four ASTM A307 bolts would provide adequate strength. Totally new to the curb system design was the concept of using two high-strength bars located transversely in the timber deck at each post location to prevent failure of the longitudinal glulam timber bridge deck.

In order to verify and, if necessary, redesign the structural components and load transfer mechanisms (the posts, glulam and curb rails, structural bolts, and high-strength bars) (Figure 1), it was necessary to determine the lateral dynamic impact loads applied to the bridge rail. Two common methods were used:

1. An approximate method to predict the lateral impact load using a mathematical model taken from *NCHRP Report 86* (*6*) and the 1977 AASHTO Barrier Guide (*7*), and
2. Computer simulation with BARRIER VII (*8*) to analyze the response of the bridge railing during impact.

The first method or mathematical model (*6,7*) is represented by Equations 1 and 2:

$$F_{\text{lat.ave.}} = \frac{M\,g\,V_I^2\sin^2\theta}{2g[\text{AL}\sin\theta - B(1-\cos\theta) + D](1{,}000)} \tag{1}$$

$$F_{\text{lat.peak}} = F_{\text{lat.ave.}} \times \text{DF} \tag{2}$$

where

- $F_{\text{lat.ave.}}$ = average lateral impact force (kN),
- $F_{\text{lat.peak}}$ = peak lateral impact force (kN),
- M = vehicle mass (2452 kg),
- V_I = impact velocity (20.1 m/sec = 72.4 km/hr),
- θ = impact angle (20 degrees),
- g = acceleration due to gravity (9.81 m/sec^2),
- AL = distance from vehicle's front end to center of mass (2.64 m),
- $2B$ = vehicle width (1.98 m),
- D = lateral displacement of railing (assumed 0 m), and
- DF = dynamic factor ($\pi/2$).

For a 2452-kg (5,400-lb) pickup impacting at 72.4 km/hr (45 mph) and 20 degrees, $F_{\text{lat.ave.}}$ and $F_{\text{lat.peak}}$ were calculated to be 69 and 109 kN (15.5 and 24.4 kips), respectively.

The second method for calculating the lateral dynamic impact force was computer simulation with BARRIER VII (*8*). Impact conditions for AASHTO PL1 tests require a 2452-kg (5,400-lb) pickup at 72.4 km/hr (45 mph) and 20 degrees. Because the BARRIER VII program has been used extensively for computer simulation modeling with 2043-kg (4,500-lb) test vehicles, a 2043-kg sedan was used instead of the 2452-kg (5,400-lb) pickup for an approximate analysis. This reduced weight necessitated an increase in either the impact angle or the impact speed in order to provide similar loading conditions. In this case, an impact with a 2043-kg (4,500-lb) sedan at 72.4 km/hr (45 mph) and 25 degrees was assumed as a conservative estimate for the impact loading. This was in agreement with the results from Equation 1. The simulation runs were conducted at a midspan location between two timber bridge-rail posts. A 1977 Plymouth Fury weighing approximately 2043 kg (4,500 lb) was selected as the simulation test vehicle.

A series of pendulum tests conducted at SwRI on 20.3 × 20.3 cm (8 × 8 in.) Douglas fir posts cantilevered 61.0 cm (24 in.) above the base revealed a peak force approximately equal to 91 kN (20.4 kips) with a failure deflection of approximately 30.0 cm (11.8 in.) (*9*). The 47.0-cm (18.5-in.) effective post height for the curb system was less than the 61.0-cm (24-in.) height used in the SwRI tests. With this reduced height and assuming a linear relationship, an increased peak force of 117 kN (26.4 kips) and a reduced failure deflection of 23.1 cm (9.1 in.) were incorporated into the computer model. The structural properties for sawn lumber posts, glulam rail, and sawn lumber curb rail are shown by Ritter et al. (*10*). Computer simulation predicted that one post would reach a maximum shear force of 125 kN (28.0 kips) and a maximum dynamic post deflection of 17.3 cm (6.8 in.).

Design of High-Strength Bars

The peak lateral impact force was calculated to be approximately 109 kN (24.4 kips) using Equation 1, and 125 kN (28.0 kips) using BARRIER VII. It was assumed that 50 percent of the 125-kN (28.0-kip) impact force was transmitted to both the upper glulam rail and the lower curb rail. The impact load acting on the curb rail was assumed to transfer directly to the bars, while the impact load acting on the upper rail was assumed to transfer to the curb rail through the post-to-curb attachment and subsequently the bars. Based on these assumptions, it was estimated that the bars placed within the deck would be required to resist a force of approximately 125 kN (28.0 kips). High-strength threaded bars complying with the ASTM A722 designation were chosen, with the smallest size available—1.6-cm (⅝-in.) diameter—with an ultimate strength of 194 kN (43.5 kips); two bars were placed through the deck panels 7.6 cm (3 in.) below the top surface of the deck at each post location. This resulted in a somewhat conservative bar design that was maintained so as to ensure a successful test with no damage to the deck. Strain gauges were placed within the ends of selected bolts and bars to determine the transmitted loads.

Timber Deck and Substructure

A full-size simulated timber bridge system was constructed at the MwRSF. To simulate an actual timber bridge installation, the longitudinal glulam timber bridge deck was mounted on six reinforced-concrete bridge supports (*10*). The inner three concrete bridge supports had center-to-center spacings of 5.72 m (18 ft 9 in.), whereas the outer two spacings were 5.56 m (18 ft 3 in.).

The longitudinal glulam timber deck consisted of 10 rectangular panels. The panels measured 1.22 m (3 ft 11⅞ in.) wide, 5.70 m (18 ft 8½ in.) long, and 27.3 cm (10¾ in.) thick. The timber deck was constructed so that two panels formed the width of the deck and five panels formed the length of the deck. The longitudinal glulam timber deck was fabricated with West Coast Douglas fir and treated with pentachlorophenol in heavy oil to a minimum net retention of 9.61 kg/m^3 (0.6 lb/ft^3) as specified in AWPA Standard C14 (*11*). At each longitudinal midspan location of the timber deck panels, stiffener beams were bolted transversely across the bottom side of the timber deck panels per AASHTO bridge design requirements. The stiffener beams measured 13.0 cm (5⅛ in.) wide, 15.2 cm (6 in.) thick, and 2.44 m (8 ft) long.

Glulam Timber and Sawn Lumber Curb Railing Design Details

The curb system consisted of four major components: sawn lumber scupper blocks, sawn lumber curb rail, sawn lumber posts, and longitudinal glulam timber rail. One timber scupper block was bolted to the timber deck at each post location with four ASTM A307 galvanized dome-head bolts 1.9 cm (¾ in.) in diameter and 61.0 cm (24 in.) long. The scupper blocks were 15.2 cm (6 in.) thick, 30.5 cm (12 in.) wide, and 0.91 m (3 ft) long and were attached to the curb rail and timber deck surface with shear plate connectors 10.2 cm (4 in.) in diameter. The sawn lumber curb rail was bolted to the top side of the scupper blocks. The nominal size of the curb rail was 15.2 cm (6 in.) deep and 30.5 cm (12 in.) wide with the top of the curb rail positioned 30.5 cm (12 in.) above the timber deck surface. One ASTM A325 dome-head bolt 3.2 cm (1¼ in.) in diameter and 55.9 cm (22 in.) long was used to attach each of the 15 bridge posts to the curb rail. Two high-strength bars 1.37 m (4 ft 6 in.) long were positioned transversely through the outer timber deck panel and spaced at 55.9 cm (22 in.) at each post. Fifteen sawn lumber Douglas fir posts measuring 20.3 cm (8 in.) wide, 20.3 cm (8 in.) deep, and 1.09 m (3 ft 6¾ in.) long and spaced at 1.90 m (6 ft 3 in.) on centers were used to support the upper glulam railing. The posts were treated to meet AWPA Standard C14 with 192.22-kg/m^3 (12-lb/ft^3) creosote (*11*). The longitudinal glulam rail was fabricated from West Coast Douglas fir and treated in the same manner as the timber deck. The glulam rail measured 17.1 cm (6¾ in.) wide and 26.7 cm (10½ in.) deep. The top mounting height of the glulam rail was 81.3 cm (2 ft 8 in.) above the timber deck surface. The glulam rail was offset from the posts with timber spacer blocks measuring 12.1 cm (4¾ in.) thick, 20.3 cm (8 in.) wide, and 26.7 cm (10½ in.) deep. Two ASTM A307 galvanized dome-head bolts 1.6 cm (⅝ in.) in diameter and 58.4 cm (23 in.) long were used to attach the glulam rail to the timber posts.

Full-Scale Crash Test

The two AASHTO PL1 crash tests conducted by SwRI (*1*) used an 825-kg (1,818-lb) minicompact at 95.3 km/hr (59.2 mph) and 20 degrees, and a 2385-kg (5,254-lb) pickup at 76.5 km/hr (47.5 mph) and 20 degrees. Because the basic geometry of the curb system was unchanged from the system tested by SwRI, it was not necessary to perform a test with a minicompact sedan. Because the structural components and load transfer mechanisms were modified, a test with a ballasted pickup was required to determine the structural adequacy and safety performance of the curb system.

Test C-1 [2,452 kg (5,400 lb), 71.0 km/hr (44.1 mph), 23.4 degrees] was conducted at the midspan location between Posts 7 and 8, which was 0.95 m (3 ft 1½ in.) upstream from a splice in the glulam rail (Figure 4). A summary of the test results and the sequential photographs are shown in Figure 5.

The vehicle became parallel to the timber bridge railing at approximately 0.273 sec with a velocity of 57.6 km/hr (35.8 mph). The vehicle exited the bridge railing at 0.434 sec at 56.4 km/hr (35.0 mph) and 5.3 degrees. The vehicle came to a stop 38.10 m (125 ft) downstream from impact. The maximum perpendicular offset to the right side of the vehicle from a line parallel to the traffic-side face of the rail was 3.10 m (10 ft 2 in.) at 24.38 m (80 ft) downstream from impact.

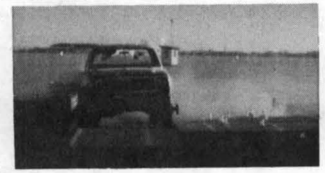
The moderate test vehicle damage is shown in Figure 4. The superficial damage to the bridge-railing system is shown in Figure 4. Minor scrapes occurred along both the upper glulam and lower curb rails. The length of vehicle contact on the upper glulam rail was approximately 2.74 m (9 ft). The lower curb rail received a gouge 15.2 cm (6 in.) long near impact. During the test, the 15.2-cm (6-in.) square steel plate washer located on the back side of Post 8 was deformed.

FIGURE 4 Impact location (*top*), vehicle damage (*middle*), and bridge-rail damage (*bottom*), Test C-1.

0.000 sec

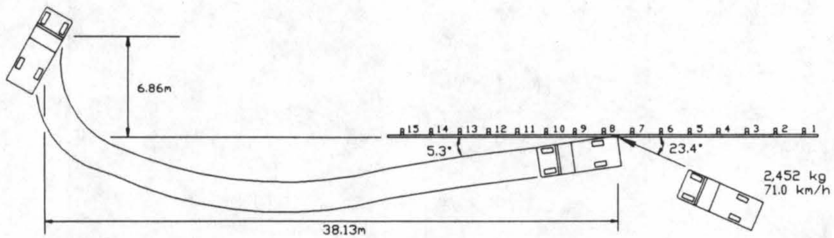
0.081 sec

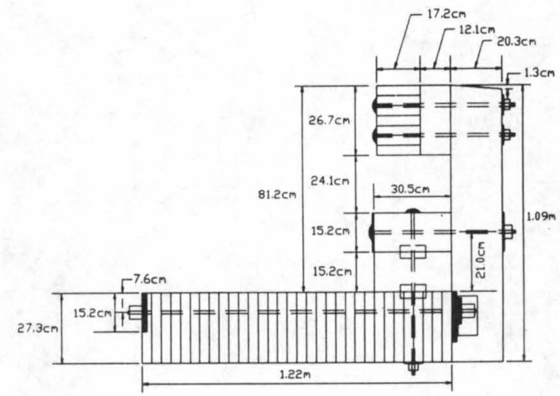
0.179 sec



0.273 sec

0.540 sec





Test Number	C-1
Date	11/16/90
Bridge Rail Installation	Glulam Timber Bridge Rail With Sawn Lumber Curb Rail
Length	28.59 m
Upper Rail	
Width	17.2 cm
Depth	26.7 cm
Top Mounting Height	81.2 cm
Material	Glulam Rail-Comb. No. 2
Lower Rail (Curb)	
Width	30.5 cm
Depth	15.2 cm
Top Mounting Height	30.4 cm
Material	Sawn Lumber
Posts (No. 1 through 15)	
Size	20.3 cm x 20.3 cm x 108.6 cm
Material	Sawn Lumber
Bridge Deck Installation	Longitudinal Glulam Timber Bridge Deck Panels
Panel Size	27.3 cm x 1.22 m x 5.72 m
Material	Glulam Timber Deck Comb. No. 2
Vehicle Model	1984 Dodge Ram
Test Inertial Weight	2,452 kg
Gross Static Weight	2,452 kg

Vehicle Speed	
Impact	71.0 km/h
Exit	56.3 km/h
Vehicle Angle	
Impact	23.4 deg
Exit	5.3 deg
Vehicle Snagging	None
Vehicle Stability	Satisfactory
Effective Coefficient of Friction (μ)	0.27 (Fair)
Occupant Impact Velocity	
Longitudinal	3.26 m/s (9.15 m/s) (2)
Lateral	4.45 m/s (7.63 m/s) (2)
Occupant Ridedown Deceleration	
Longitudinal	2.3 g's (15 G's) (2)
Lateral	9.6 g's (15 G's) (2)
Vehicle Damage	Moderate
TAD	1-RFQ-4
VDI	01RFEW2
Vehicle Rebound Distance	3.1 m @ 24.4 m
Bridge Rail Damage	Minor Scrapes on Rail and Curb
Maximum Post Deflection	
Permanent Set	0 m
Dynamic	13.2 cm

FIGURE 5 Summary of test results and sequential photographs, Test C-1.

Analysis of the available strain gauge data indicated that the actual loads transmitted to the connecting bolts and bars were lower than anticipated. The strain gauge located in an ASTM A722 bar 1.6 cm (⅝ in.) in diameter at Post 8 transmitted a load of approximately 28 kN (6.2 kips). For two bars, the load transferred to the deck would total approximately 55 kN (12.4 kips). The strain gauge located in the ASTM A325 bolt 3.2 cm (1¼ in.) in diameter at Post 9 carried a load of approximately 26 kN (5.9 kips). The actual loads carried by the structural bolts and bars were much less than the design loads, because the original design assumed the entire lateral load would not be distributed to more than one post, whereas the load was actually distributed over several posts.

SHOE-BOX SYSTEM

System Development

The single longitudinal glulam timber bridge railing or shoe-box system was developed to obtain an alternate glulam bridge railing without a curb. However, a single bridge-rail (side-mounted) design requires larger rail and post members, since the impact force is transferred to the single rail element. Posts with greater flexural stiffness were required to compensate for this increased moment. A sawn lumber post 20.3 cm (8 in.) wide and 25.4 cm (10 in.) deep was chosen for the initial computer simulation analysis; however, it was determined that an S4S dressed lumber post provided better quality control of cross-sectional dimensions. This control was critical to ensure correct placement of posts in the structural steel shoe-box support. The S4S dressed lumber post measured 19.0 cm (7½ in.) wide and 24.1 cm (9½ in.) deep. A single glulam rail 17.1 cm (6¾ in.) wide and 34.3 cm (13½ in.) deep was also selected (*5*). The structural properties for the sawn lumber posts and glulam rail are given by Ritter et al. (*10*). The glulam rail was determined to provide adequate moment capacity while also providing an adequate maximum clearance of 47.0 cm (18½ in.) from the top of the deck surface to the bottom of the glulam rail. For traffic railings, AASHTO (*4*) states that a maximum clear opening below the bottom rail shall not exceed 43.2 cm (17 in.), but with the rail blocked out 31.8 cm (12½ in.), the authors determined that with an 817-kg (1,800-lb) car, vehicle snagging would not occur on the post during impact.

It was necessary to determine the increased lateral dynamic impact loads applied to the bridge rail in the shoe-box system (Figure 2) to design the high-strength bars used to transfer the impact force into the timber deck and to verify the structural adequacy of the rail and post elements. The mathematical model (*6,7*) and computer simulation with BARRIER VII (*8*) were again used for the analysis. Using the mathematical model with a 2452-kg (5,400-lb) pickup impacting at 72.4 km/hr (45 mph) and 20 degrees and an assumed lateral railing displacement equal to 0.15 m (0.5 ft), $F_{lat.ave.}$ and $F_{lat.peak}$ were calculated to be 58 kN (13.1 kips) and 92 kN (20.6 kips), respectively. Computer simulation with BARRIER VII (*8*) was used with the same vehicle and impact conditions as for the curb system. The analysis of the simulation results revealed a maximum post shear force of 82 kN (18.5 kips) and a maximum dynamic deflection of 19.3 cm (7.6 in.).

Design of High-Strength Bars

The peak lateral impact force was calculated to be approximately 92 kN (20.6 kips) (*6,7*) and 82 kN (18.5 kips) (*8*). Considering the compressive bearing force between the sawn lumber post and the steel shoe box, it was estimated that the bars placed within the deck would be required to resist a force of approximately 485 kN (109.1 kips). High-strength threaded bars complying with the ASTM A722 designation were chosen. The next available size of threaded bar larger than 1.6 cm (⅝ in.) was 2.5 cm (1 in.) in diameter with an ultimate strength of 567 kN (127.5 kips). Two bars were placed through the deck panels 7.6 cm (3 in.) below the top surface of the deck. This resulted in a somewhat conservative bar design that was maintained so as to ensure a successful test with no damage to the deck. Strain gauges were placed within the ends of selected bars to determine the loads transmitted, and if necessary, redesign the hardware.

Single Glulam Timber Bridge-Railing Design Details

The concrete substructure used to support the timber deck for the curb system was maintained for the shoe-box system. However, the two rows of timber deck panels (five panels per row) were interchanged for the new bridge-rail configuration.

The shoe-box system consisted of three major components: dressed lumber posts, longitudinal glulam timber rail, and structural steel shoe boxes. The glulam railing was supported with 15 timber posts spaced 1.90 cm (6 ft 3 in.) on centers. The S4S dressed lumber posts measured 19.0 cm (7½ in.) wide, 24.1 cm (9½ in.) deep, and 1.09 m (3 ft 6¾ in.) long. The posts were treated with creosote to 192.22 kg/m³ (12 lb/ft³) to meet AWPA Standard C14 (*11*). The timber posts were attached to the longitudinal glulam timber deck with ASTM A36 structural steel shoe boxes. Fifteen welded-steel shoe boxes were fabricated with a steel plate 2.5 cm (1 in.) thick, 27.3 cm (10¾ in.) wide, and 61 cm (24 in.) long for the bearing surface with the remaining three sides of the box fabricated with 1.3-cm (½-in.) steel plate. A galvanized nail spike 0.6 cm (¼ in.) in diameter and 15.2 cm (6 in.) long was driven into the post through a hole located on the back side of the steel shoe box to prevent post pullout. Two high-strength bars 1.37 m (4 ft 6 in) long were positioned transversely through the outer timber deck panel and spaced at 40.6 cm (16 in.) at each post. The longitudinal glulam rail was fabricated from West Coast Douglas fir and treated in the same manner as the timber deck. The glulam rail measured 17.1 cm (6¾ in.) wide and 34.3 cm (13½ in.) deep. The top mounting height of the glulam rail was 81.3 cm (2 ft 8 in.) above the timber deck surface. The glulam rail was offset from the posts with timber spacer blocks measuring 14.6 cm (5¾ in.) thick, 20.3 cm (8 in.) wide, and 34.3 cm (13½ in.) deep. Two ASTM A307 galvanized dome-head bolts 1.6 cm (⅝ in.) in diameter and 61.0 cm (24 in.) long were used to attach the glulam rail to the timber posts.

Full-Scale Crash Tests

Test FSSB-1

Test FSSB-1 [2452 kg (5,400 lb), 72.4 km/hr (45.0 mph), 21.8 degrees] was conducted at the midspan location between Posts

7 and 8, which was 0.95 m (3 ft 1½ in.) upstream from a splice in the glulam rail (Figure 6). A summary of the test results and the sequential photographs are shown in Figure 7.

The vehicle became parallel to the bridge railing at 0.282 sec with a velocity of 62.0 km/hr (38.5 mph). The vehicle exited the bridge railing at 0.456 sec at 57.0 km/hr (35.4 mph) and 7.5 degrees. The vehicle came to a stop 30.5 m (100 ft) downstream from impact. The maximum perpendicular offset to the right side of the vehicle from a line parallel to the traffic-side face of the rail was 0.61 m (2 ft).

The moderate test vehicle damage is shown in Figure 6. The superficial damage to the bridge rail is shown in Figure 6. Both minor scrapes and gouging were evident along the traffic-side face of the bridge rail. Post 8 received some indentations and black marks on the upstream exposed corner; evidence of tire marks on the timber deck near Post 8 are shown in Figure 6. The gouging on the lower portion of the rail occurred from the minor snagging with the right-front wheel well and right-door panel joint. Additional gouging occurred to the top of the timber deck surface. The lower-rear side of Post 8 was compressed 0.3 cm (⅛ in.) due to bearing against the steel shoe box.

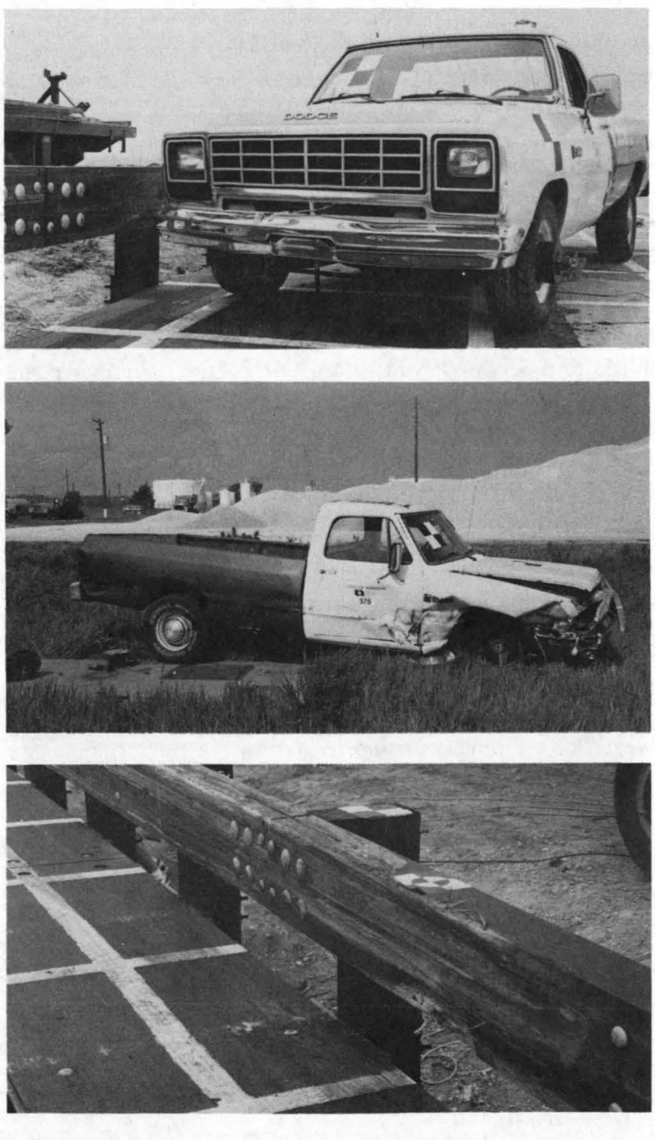


FIGURE 6 Impact location (*top*), vehicle damage (*middle*), and bridge-rail damage (*bottom*), Test FSSB-1.

The analysis of the strain gauge data indicated that the actual loads transmitted to the high-strength bars were less than anticipated. The maximum load transmitted to an ASTM A722 bar was 164 kN (36.8 kips). The total combined load for the two bars at Post 8 was 301 kN (67.7 kips). A comparison between total load carried by the bars per post location and the maximum lateral dynamic deflection showed significant correlation (i.e., the maximum lateral dynamic post deflection corresponded to the maximum load carried in two bars at a post location) (*10*).

Test FSSB-2

Test FSSB-2 [839 kg (1,849 lb), 80.7 km/hr (50.1 mph), 21.5 degrees] was conducted at the midspan location between Posts 5 and 6, which was 0.95 m (3 ft 1½ in.) upstream from the centerline of Post 6 (Figure 8). A summary of the test results and the sequential photographs are shown in Figure 9.

The vehicle became parallel to the bridge railing at 0.204 sec with a velocity of 63.3 km/hr (39.3 mph). The vehicle exited the bridge railing at 0.442 sec at 60.9 km/hr (37.8 mph) and 6.7 degrees. The vehicle came to a stop 106.68 m (350 ft) downstream from impact. The maximum perpendicular offset to the right side of the vehicle from a line parallel to the traffic side face of the rail was 3.96 m (13 ft).

The minimal test vehicle damage is shown in Figure 8. The superficial damage to the bridge rail is shown in Figure 8. Both minor scrapes and gouging were evident along the traffic-side face of the bridge rail.

STEEL SYSTEM

System Development

A single steel thrie-beam bridge railing, or steel system (Figure 3) was developed to obtain a more economical AASHTO PL1 bridge railing for timber decks. The two previously tested glulam bridge railings had higher initial material costs per foot than the steel system. The development of an AASHTO PL1 steel railing for timber decks began with a literature review of existing steel-rail bridge railings. Information gathered in the review suggested that side-mounted steel systems such as the NCHRP SL1 thrie beam (*12*), the Oregon side-mounted thrie-beam bridge rail (PL1) (*13,14*), the California Type 115 bridge rail (PL1) (*15*), and the California thrie-beam bridge rail (PL1) (*15*) could be modified for timber bridge decks. After reviewing data on the steel side-mounted systems, the California thrie-beam bridge rail was selected for modification to the timber bridge deck.

The steel system was attached to the longitudinal glulam timber deck with high-strength bars as previously used in the shoe-box system. Strain gauges were placed within the ends of the bars to determine the loads transmitted.

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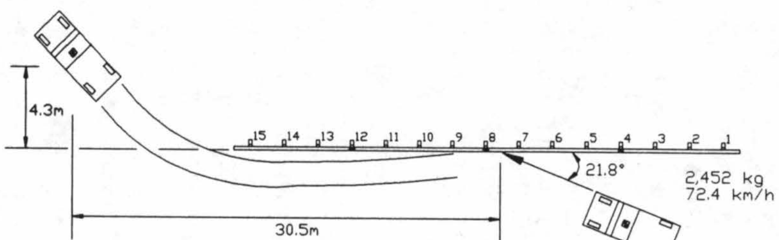
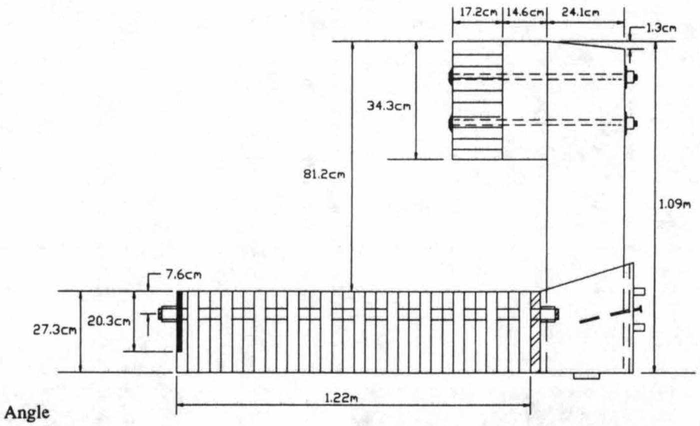




Test Number	FSSB-1
Date	5/14/91
Bridge Rail Installation	Glulam Timber Bridge Rail With Steel Shoe Box
Length	28.59 m
Glulam Timber Rail	
Width	17.2 cm
Depth	34.3 cm
Top Mounting Height	81.2 cm
Material	Glulam Rail-Comb. No. 2
Posts (No. 1 through 15)	
Size	19.0 cm x 24.1 cm x 108.6 cm
Material	S4S Dressed Lumber
Bridge Deck Installation	Longitudinal Glulam Timber Bridge Deck Panels
Panel Size	27.3 cm x 1.22 m x 5.72 m
Material	Glulam Timber Deck Comb. No. 2
Vehicle Model	1984 Dodge Ram
Test Inertial Weight	2,452 kg
Gross Static Weight	2,452 kg
Vehicle Speed	
Impact	72.4 km/h
Exit	57.0 km/h

Vehicle Angle	
Impact	21.8 deg
Exit	7.5 deg
Vehicle Snagging	Minor Wheel Well and Door Joint gouging
Vehicle Stability	Satisfactory
Effective Coefficient of Friction (μ)	0.20 (Good)
Occupant Impact Velocity	
Longitudinal	3.42 m/s (9.15 m/s) (2)
Lateral	5.19 m/s (7.63 m/s) (2)
Occupant Ridedown Deceleration	
Longitudinal	2.0 g's (15 g's) (2)
Lateral	2.5 g's (15 g's) (2)
Vehicle Damage	Moderate
TAD	1-RFQ-3
VDI	01RFEW2
Maximum Vehicle Rebound Distance	0.61 m
Bridge Rail Damage	Minor Scrapes and Gouging on the Rail
Maximum Dynamic Deflection	
Rail	19.6 cm (Visible)
Post	23.6 cm

FIGURE 7 Summary of test results and sequential photographs, Test FSSB-1.

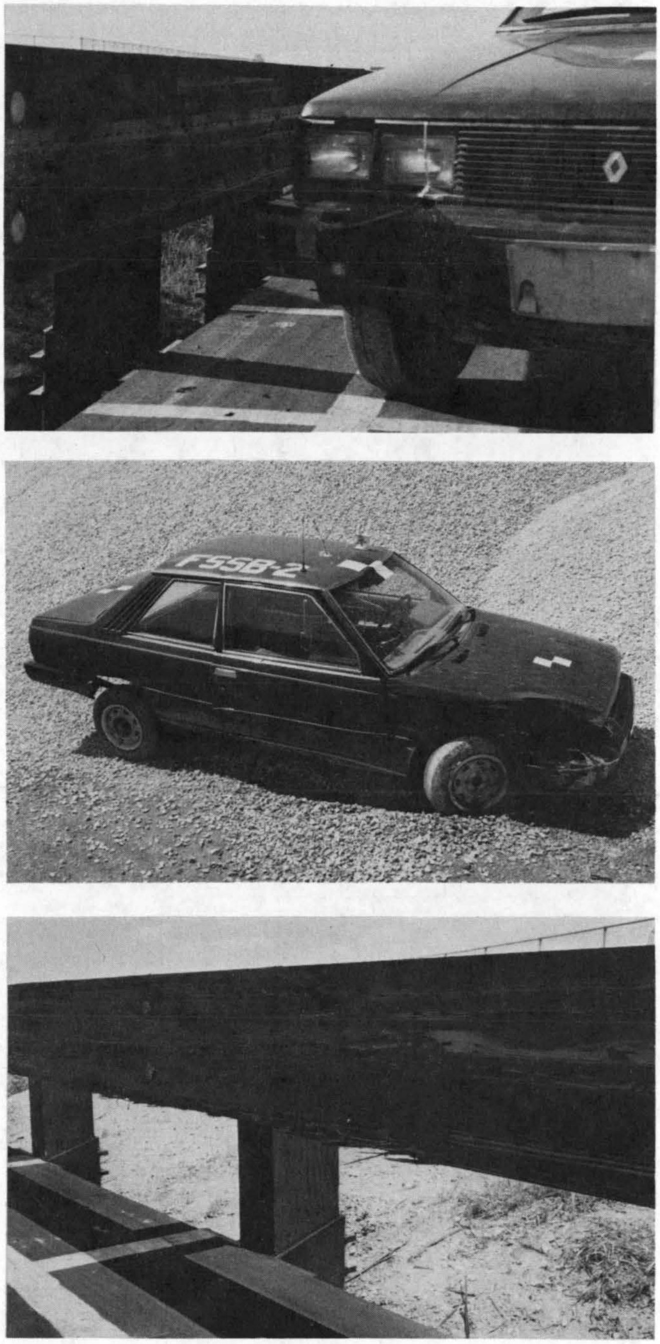


FIGURE 8 Impact location (*top*), vehicle damage (*middle*), and bridge-rail damage (*bottom*), Test FSSB-2.

Single Steel Thrie-Beam Railing Design Details

The concrete substructure used to support the timber deck for the curb and shoe-box systems was maintained for the steel system, as was the configuration of the timber deck panels used during the testing of the shoe-box system.

The steel system consisted of three major components: structural steel posts, steel thrie-beam rail, and structural steel mounting plates. The 10-gauge steel thrie beam was supported by 15 ASTM A36 W6 × 15 structural steel posts measuring 93.3 cm (3 ft ¾ in.) long. The steel posts were attached to the longitudinal glulam timber deck with ASTM A36 structural steel mounting plates. Fifteen steel mounting plates measuring 1.9 cm (¾ in.) thick, 27.3 cm (10¾ in.) deep, and 61 cm (24 in.) long were attached to the deck with two ASTM A722 high-strength bars 2.5 cm (1 in.) in diameter and 137.2 cm (4 ft 6 in.) long spaced at 40.64 cm (16 in.) and located 7.6 cm (3 in.) below the top surface of the deck. Each steel post was bolted to a steel mounting plate with four ASTM A325 galvanized hex-head bolts 2.2 cm (⅞ in.) in diameter that were welded to the deck side of the steel plate. Four recessed holes were cut into the edge of the timber deck so that the steel mounting plate could be bolted flush against the vertical deck surface. The top mounting height of the thrie-beam rail was 78.4 cm (2 ft 6⅞ in.) above the timber deck surface. A 78.4 cm (2 ft 6⅞ in.) mounting height was selected in order to maintain compatibility with the transition section between the W-beam and the thrie-beam. The steel thrie-beam rail was offset from the posts with ASTM A36 W6 × 15 structural steel spacer blocks measuring 56.2 cm (1 ft 10⅛ in.) long.

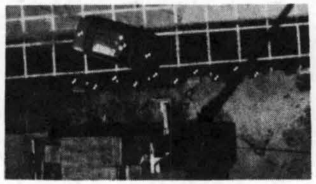
Full-Scale Crash Test

The California thrie-beam bridge rail 81.3 cm (32 in.) high successfully met AASHTO PL1 (*2*) and NCHRP 230 (*3*) requirements for structural adequacy, occupant risk, and vehicle trajectory (*15*). The geometry of the California thrie-beam bridge rail was basically unchanged with only a slight reduction in the effective railing height. It was not necessary to perform a test with an 817-kg (1,800-lb) vehicle impacting at 80.5 km/hr (50 mph) and 20 degrees, since there was no potential for wheel snagging or concern for occupant risk. The only concern with using an existing steel railing system was to verify the structural adequacy of the load transfer mechanism in the post-to-deck connection, which could be determined with a 2452-kg (5,400-lb) vehicle impacting at 72.4 km/hr (45 mph) and 20 degrees.

Test FSSR-1 [2542 kg (5,600 lb), 71.2 km/hr (44.2 mph), 19.1 degrees] was conducted at the midspan location between Posts 7 and 8, which was 0.95 m (3 ft 1½ in.) upstream from a central splice in the thrie-beam rail (Figure 10). A summary of the test results and sequential photographs are shown in Figure 11.

The vehicle became parallel to the bridge rail at 0.240 sec with a velocity of 57.3 km/hr (35.6 mph). The unrestrained onboard dummy impacted the right-side window at 0.291 sec. The vehicle exited the bridge railing at 0.401 sec at approximately 57.3 km/hr (35.6 mph) and 13.3 degrees. During vehicle redirection, the dummy launched out of the right-side window. The vehicle came to a stop 30.78 m (101 ft) downstream from the impact point. The maximum perpendicular offset to the right side of the vehicle from a line parallel to the traffic-side face of the rail was 1.78 m (5 ft 10 in.) at 18.29 m (60 ft) downstream from impact.

The minor test vehicle damage is shown in Figure 10. The moderate bridge-rail and post damage is shown in Figure 10. The physical damage to the thrie beam, consisting of scrapes, tire marks, and deformation, was measured to be 3.94 m (12 ft 11 in.) long.



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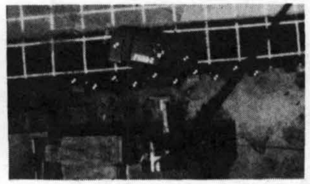

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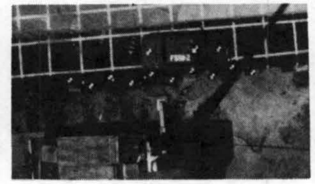

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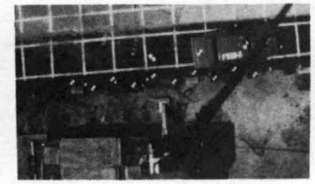

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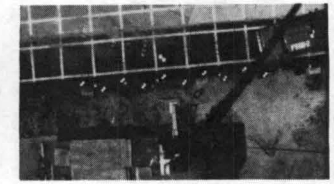

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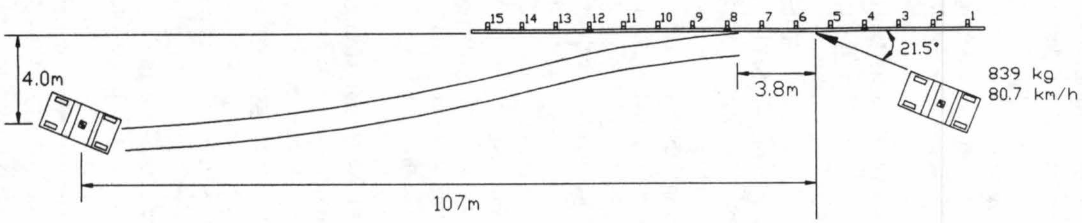

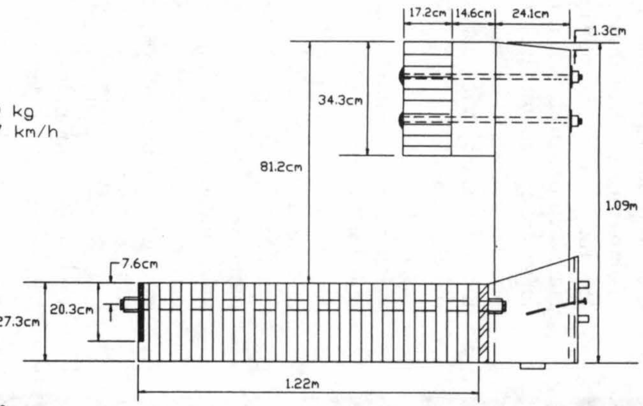

Test Number	FSSB-2
Date	6/12/91
Bridge Rail Installation	Glulam Timber Bridge Rail With Steel Shoe Box
Length	28.59 m
Glulam Timber Rail	
Width	17.2 cm
Depth	34.3 cm
Top Mounting Height	81.2 cm
Material	Glulam Rail-Comb. No. 2
Post (No. 1 through 15)	
Size	19.0 cm x 24.1 cm x 108.6 cm
Material	S4S Dressed Lumber
Bridge Deck Installation	Longitudinal Glulam Timber Bridge Deck Panels
Panel Size	27.3 cm x 1.22 m x 5.72 m
Material	Glulam Timber Deck Comb. No. 2
Vehicle Model	1984 Renault Alliance
Test Inertial Weight	839 kg
Gross Static Weight	839 kg
Vehicle Speed	
Impact	80.7 km/h
Exit	60.9 km/h

Vehicle Angle	
Impact	21.5 deg
Exit	6.7 deg
Vehicle Snagging	None
Vehicle Stability	Satisfactory
Effective Coefficient of Friction (μ)	0.40 (Marginal)
Occupant Impact Velocity	
Longitudinal	4.54 m/s (9.15 m/s) (2)
Lateral	6.59 m/s (7.63 m/s) (2)
Occupant Ridedown Deceleration	
Longitudinal	1.1 g's (15 g's) (2)
Lateral	6.5 g's (15 g's) (2)
Vehicle Damage	Minimal
TAD	1-RFQ-3
VDI	01RFEW1
Vehicle Rebound Distance	3.97 m
Bridge Rail Damage	Minor Scrapes and Gouging on the Rail
Maximum Dynamic Deflection	
Rail	11.7 cm
Post	11.2 cm

FIGURE 9 Summary of test results and sequential photographs, Test FSSB-2.

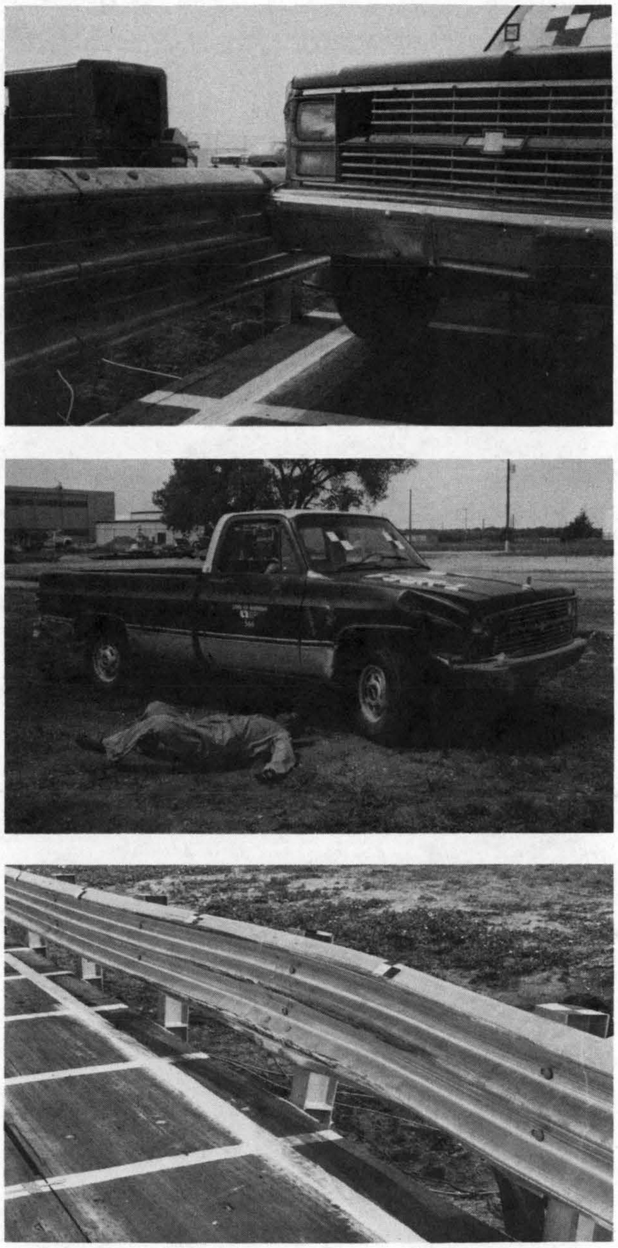

FIGURE 10 Impact location (*top*), vehicle damage (*middle*), and bridge-rail damage (*bottom*), Test FSSR-1.

The analysis of the strain gauge data revealed that the maximum load transmitted to an ASTM A722 bar was 186 kN (41.8 kips). For two bars, the load transferred to the deck would total approximately 372 kN (83.6 kips).

RAILING COMPARISONS

Three AASHTO PL1 bridge railings were developed: two glulam timber railing systems and one steel railing system. All three bridge railings received acceptable safety performance evaluations according to the AASHTO guidelines (*2*), but the following comparisons can be made.

After each crash test, the examination of the top and bottom surfaces of the timber deck laminations revealed that there was no physical damage or separation. Therefore, the maintenance costs of these three railing systems should be less than that of the previously tested system on a spike-laminated timber deck (*1*). In addition, all three bridge railings were easy to install; thus, they should have low construction labor costs. The material costs for the curb system, the shoe-box system, and the steel system were approximately \$325/m (\$99/ft), \$213/m (\$65/ft), and \$138/m (\$42/ft), respectively.

Permanent structural damage as a result of impact occurred only for the steel system. The significant damage (plastic deformation) occurred to 3.94 m (12 ft 11 in.) of thrie-beam rail and three steel posts and mounting plates. This amount of damage would produce higher maintenance costs than for the other two systems. The post and rail members responded in an elastic manner with no measurable permanent set for the curb system and shoe-box system. Damage to the glulam railings for both of these systems was mostly superficial. Therefore, the glulam rails would remain functional from a structural adequacy point of view. From an aesthetical point of view, the glulam rails could be rotated 180 degrees so as to hide the superficial damage from view. Even after the glulam railing surfaces had been scraped, shredded, and exposed to the environment, the preservative treatment came to the surface, essentially providing future protection.

CONCLUSIONS AND RECOMMENDATIONS

The safety performance evaluations of the curb system, the shoe-box system, and the steel system were acceptable according to the AASHTO PL1 guidelines (*2*). Therefore, three crashworthy bridge-railing systems were developed and are recommended for use on longitudinal timber bridge decks. Although the three bridge-railing systems were tested on a longitudinal glulam timber bridge deck, the three systems could be adapted to other longitudinal timber bridge decks. In addition, the development of the three systems addressed the concerns for aesthetics and economy. The curb system satisfied the concern for aesthetics, whereas the shoe-box system was aesthetically pleasing and more economical than the curb system. The steel system satisfied the basic need for developing an economical railing system for timber decks.

While the strain gauge results for the curb system indicated that one bar would provide sufficient strength, the use of two ASTM A722 high-strength bars 1.6-cm (⅝-in.) in diameter, or similar bars of comparable strength, is recommended for each post location. For economic considerations, additional research is suggested to redevelop the curb system using only one high-strength bar per post with smaller scupper blocks and less structural steel hardware. This redesign would reduce material costs by 14 percent to \$279/m (\$85/ft). The strain gauge results for the ASTM A325 3.2-cm (1¼-in) diameter bolt at the curb-to-post connection revealed that the load was not sufficient to warrant the use of high-strength material. Therefore, it is recommended that the design be modified to include an ASTM A307 3.18-cm (1¼-in.) diameter bolt. Similarly, it is recommended that two ASTM A722 high-strength bars 2.5-cm (1-in.) diameter or similar bars of comparable

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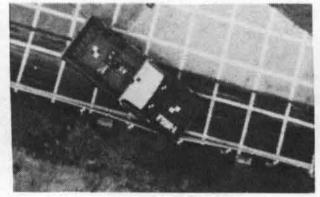

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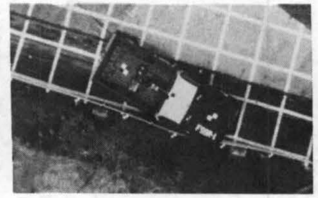

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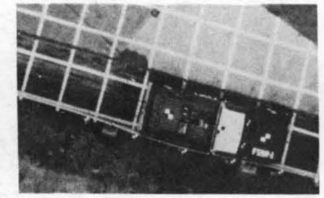

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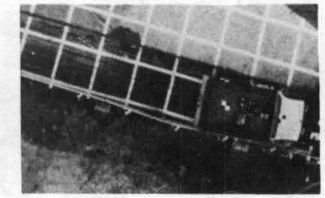

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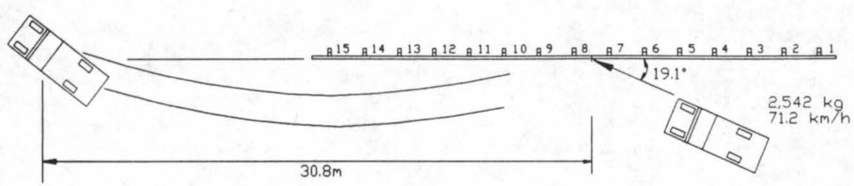

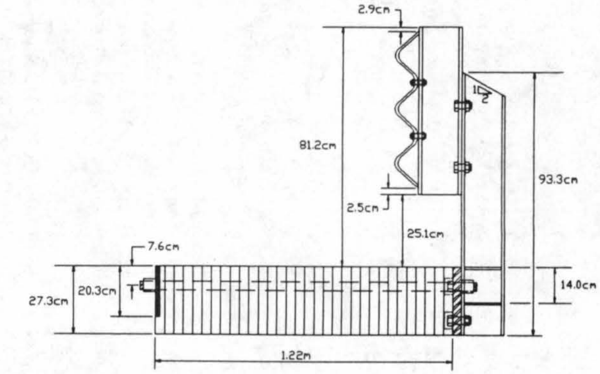

Test Number	FSSR-1
Date	6/4/92
Bridge Rail Installation	PL-1 Steel Rail with Steel Posts
Total Length	28.59 m
Steel Thrie Beam Rail	
Size	10 Gauge
Top Mounting Height	78.3 cm
Steel Posts (No. 1 through 15)	W6 x 15
Length	93.3 cm
Steel Spacer Blocks (No. 1 through 15)	W6 x 15
Length	56.1 cm
Vehicle Model	1984 Chevrolet Custom Deluxe 20
Test Inertial Weight	2,542-kg
Gross Static Weight	2,617-kg
Vehicle Speed	
Impact	71.2 km/h
Exit	57.3 km/h
Vehicle Angle	
Impact	19.1 deg
Exit	13.3 deg

Vehicle Snagging	None
Vehicle Stability	Satisfactory
Effective Coefficient of Friction (μ)	0.43 (Marginal)
Occupant Impact Velocity	
Longitudinal	4.09 m/s (9.15 m/s) (2)
Lateral	5.98 m/s (7.63 m/s) (2)
Occupant Ridedown Deceleration	
Longitudinal	-2.7 g's (15 g's) (2)
Lateral	11.4 g's (15 g's) (2)
Vehicle Damage	Minor
TAD	1-RFQ-3
VDI	01RFEW2
Bridge Rail Damage	Moderate
Maximum Vehicle Rebound Distance	1.77 m @ 18.3 m
Maximum Permanent Set Deflection	
Rail	18.8 cm
Post	20.6 cm
Maximum Dynamic Deflection	
Rail	32.0 cm
Post	35.1 cm

FIGURE 11 Summary of test results and sequential photographs, Test FSSR-1.

strength be maintained at each post location for the shoe-box system and the steel system.

ACKNOWLEDGMENTS

The authors would like to thank the following organizations who have contributed to the success of this research project: the American Institute of Timber Construction, Vancouver, Washington, for donating the glulam materials for the deck and rail construction; Laminated Concepts, Inc., Elmira, New York, for donating structural hardware for the bridge rails; Western Wood Structures, Inc., Tualatin, Oregon, for drafting of preliminary designs and shop drawings; Midwest Transportation Center, Ames, Iowa, for graduate student support; and the Office of Sponsored Programs and the Center for Infrastructure Research, University of Nebraska-Lincoln, for matching support.

DEDICATION

This research project is dedicated to Edward R. Post (1934–1991), former Professor of Civil Engineering at the University of Nebraska-Lincoln and founder and Director of MwRSF. The development of crashworthy timber bridge railings was one of his last active research projects.

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