

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

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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

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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 (¾ 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.

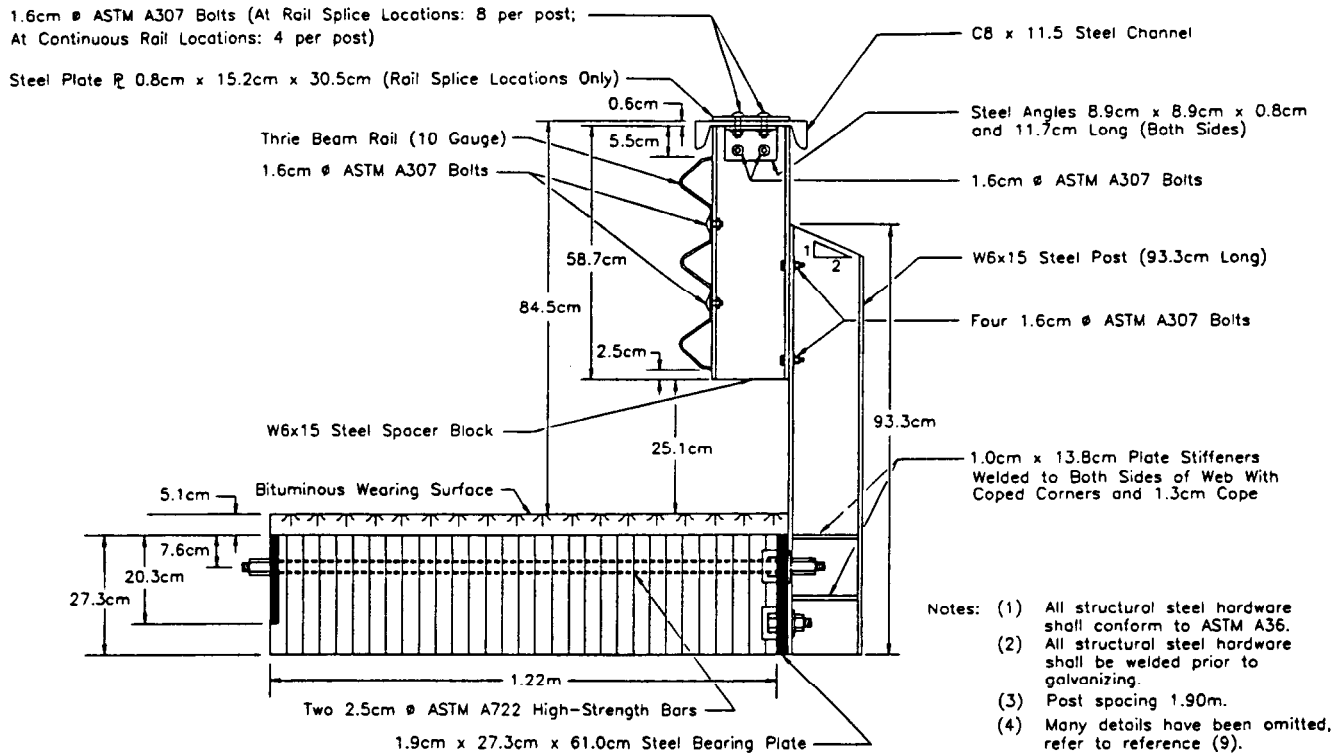


FIGURE 1 Thrie-beam with channel bridge railing (TBC-8000).

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/m<sup>3</sup> (0.6 lb/ft<sup>3</sup>) 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-

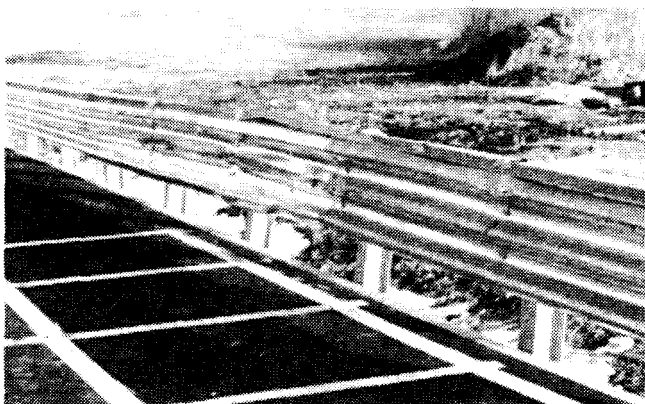
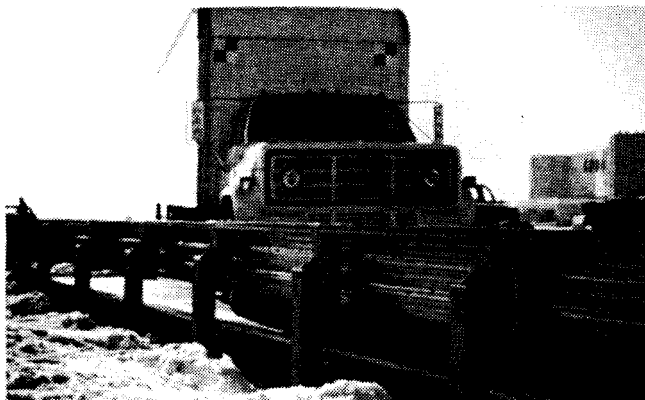
ing. The simulation was conducted modeling a 8,165-kg (18,000-lb) single-unit truck striking the rail at 80.5 km/hr (50 mph) and 15 degrees.

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.

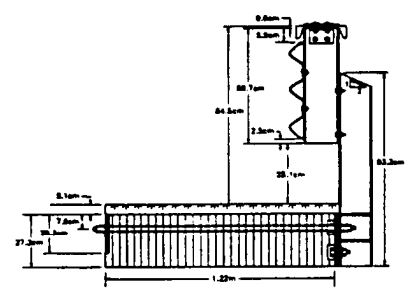
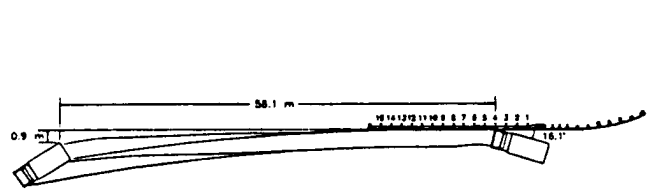
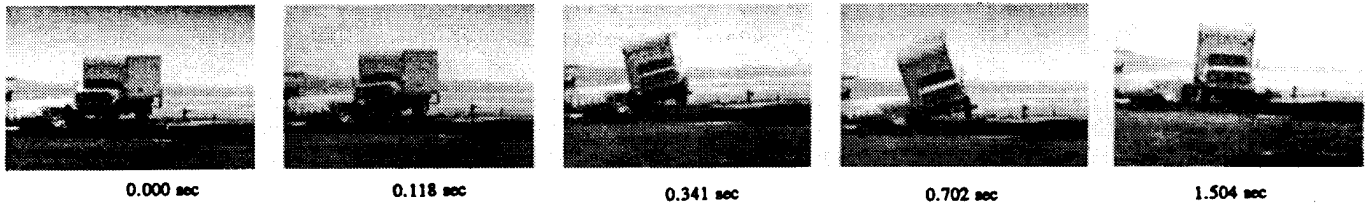
The glulam rail previously tested at SwRI (3) was crash-tested using an 825-kg (1,818-lb) minicompact at 95.3 km/hr (59.2 mph) and 20 degrees and a 2,383-kg (5,254-lb) pickup at 76.4 km/hr (47.5 mph) and 20 degrees. Because the basic geometry of the PL1 curb system and the newly developed GC-8000 were 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



Test Number	FSTC-1
Date	12/9/92
Bridge Rail Installation	Steel Thrie and Channel Bridge Rail
Total Length	28.57 m
Steel Thrie Beam Rail	
Size	10 Gauge (AASHTO M180)
Top Mounting Height	78.4 cm
Steel Channel Rail (C-rail)	
Size	C8 x 11.5 (A36)
Top Mounting Height	84.5 cm
Steel Posts (No. 1 through 15)	W6 x 15 (A36)
Length	93.3 cm
Steel Spacer Blocks (No. 1 through 15)	W6 x 15 (A36)
Length	58.7 cm
Vehicle Model	1986 GMC 7000 Series Straight Truck
Test Inertial Weight	8,165 kg
Gross Static Weight	8,165 kg
Vehicle Speed	
Impact	76.3 km/hr
Exit	Not Available

Vehicle Angle	
Impact	16.1 degrees
Exit	1.8 degrees
Vehicle Snagging	None
Vehicle Stability	Satisfactory
Effective Coefficient of Friction ( $\mu$ )	0.31
Normalized Occupant Impact Velocity	
Longitudinal (Not Required)	3.3 m/s (9.1 m/s) (4)
Lateral (Not Required)	4.8 m/s (7.6 m/s) (4)
Occupant Ridsdown Deceleration	
Longitudinal (Not Required)	1.8 g's (15 g's) (4)
Lateral (Not Required)	6.1 g's (15 g's) (4)
Vehicle Damage	Minor
TAD	1-RPQ-2
VDI	01RYEW1
Bridge Rail Damage	Moderate
Maximum Vehicle Rebound Distance	17.8 cm @ 40.54 m
Maximum Permanent Set Deflection	
C-Rail	19.3 cm
Thrie Beam	20.8 cm

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

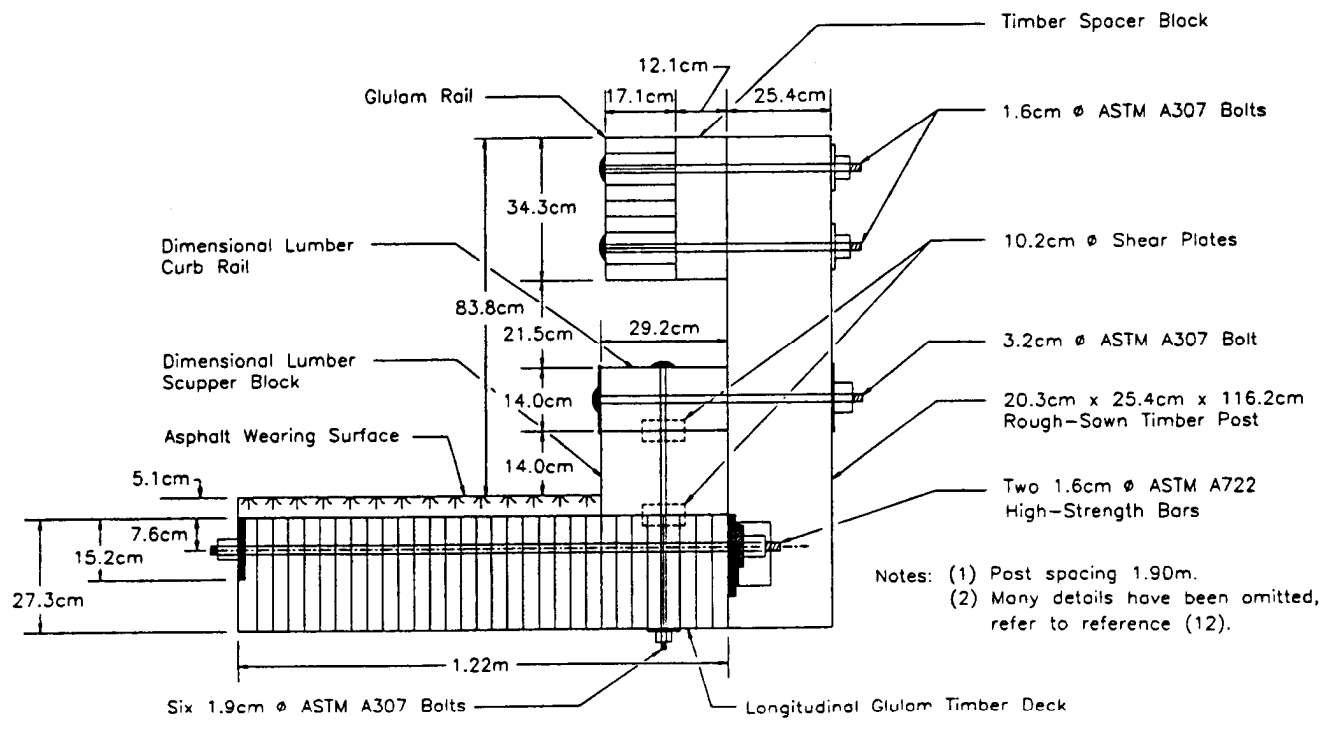


FIGURE 4 Glulam timber with curb bridge railing (GC-8000).

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/m<sup>3</sup> (12 lb/ft<sup>3</sup>) 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

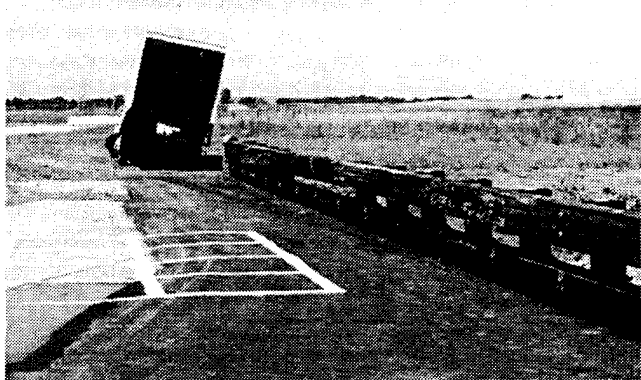
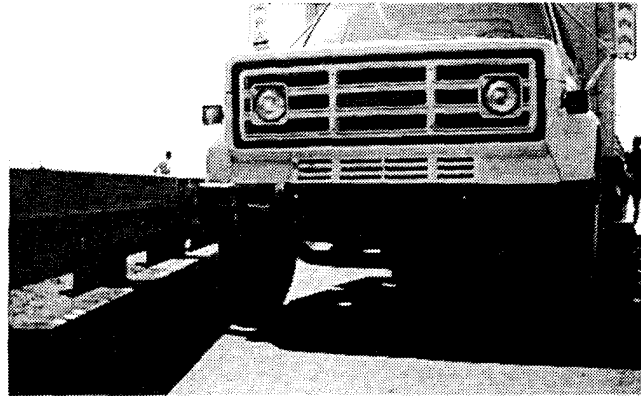
After the preliminary design of the GC-8000, computer simulation modeling with BARRIER VII (11) was performed to analyze the dynamic performance of the bridge railing before full-scale crash testing. Computer simulations were conducted with an 8,165-kg (18,000-lb) single-unit truck hitting the rail at 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.

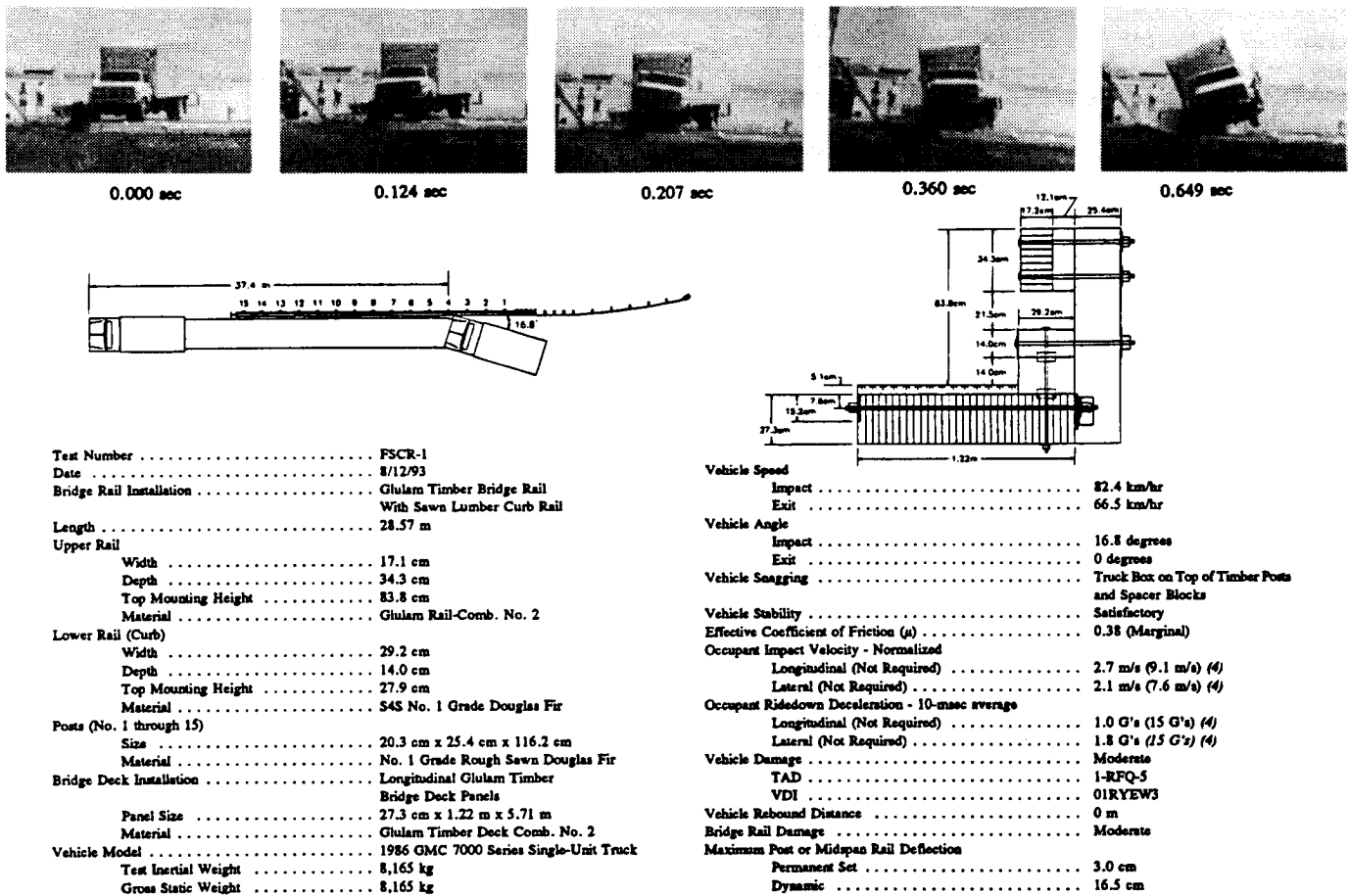


FIGURE 6 Summary of test results and sequential photographs, Test FSCR-1.

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.).

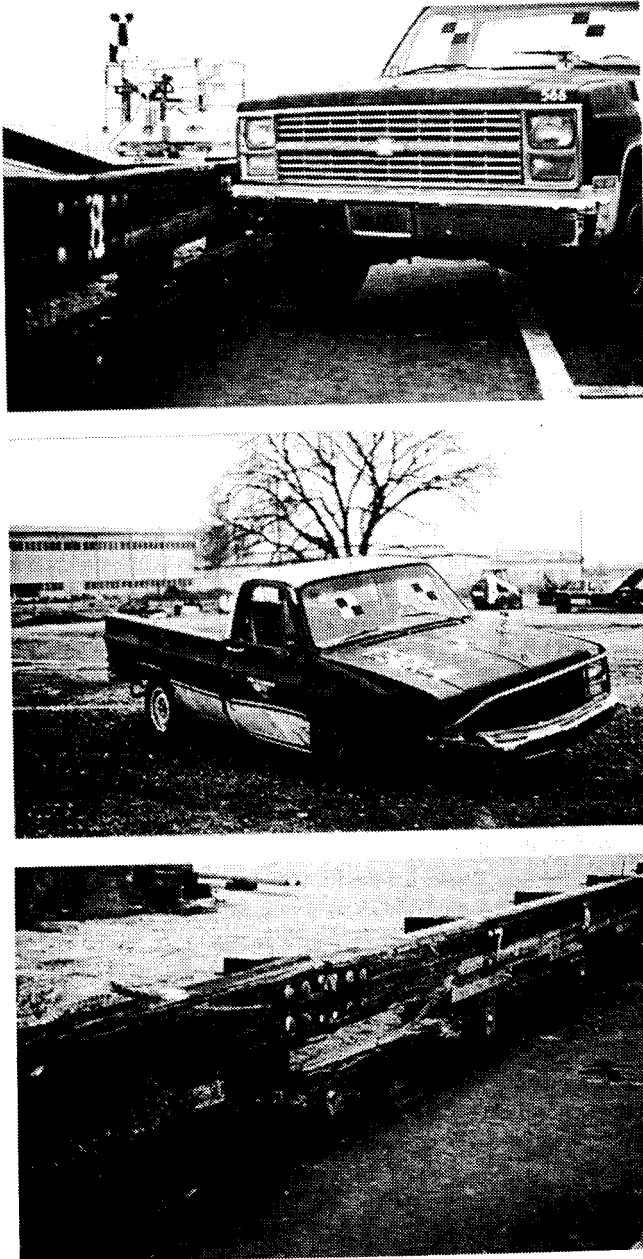
The maximum lateral permanent set deflections for midspan rail and post locations, as determined from field measurements in the impact region, were approximately 3.0 cm (1.2 in.) and 2.8 cm (1.1 in.), respectively. The maximum dynamic lateral deflections for midspan rail and post locations (determined 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

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*



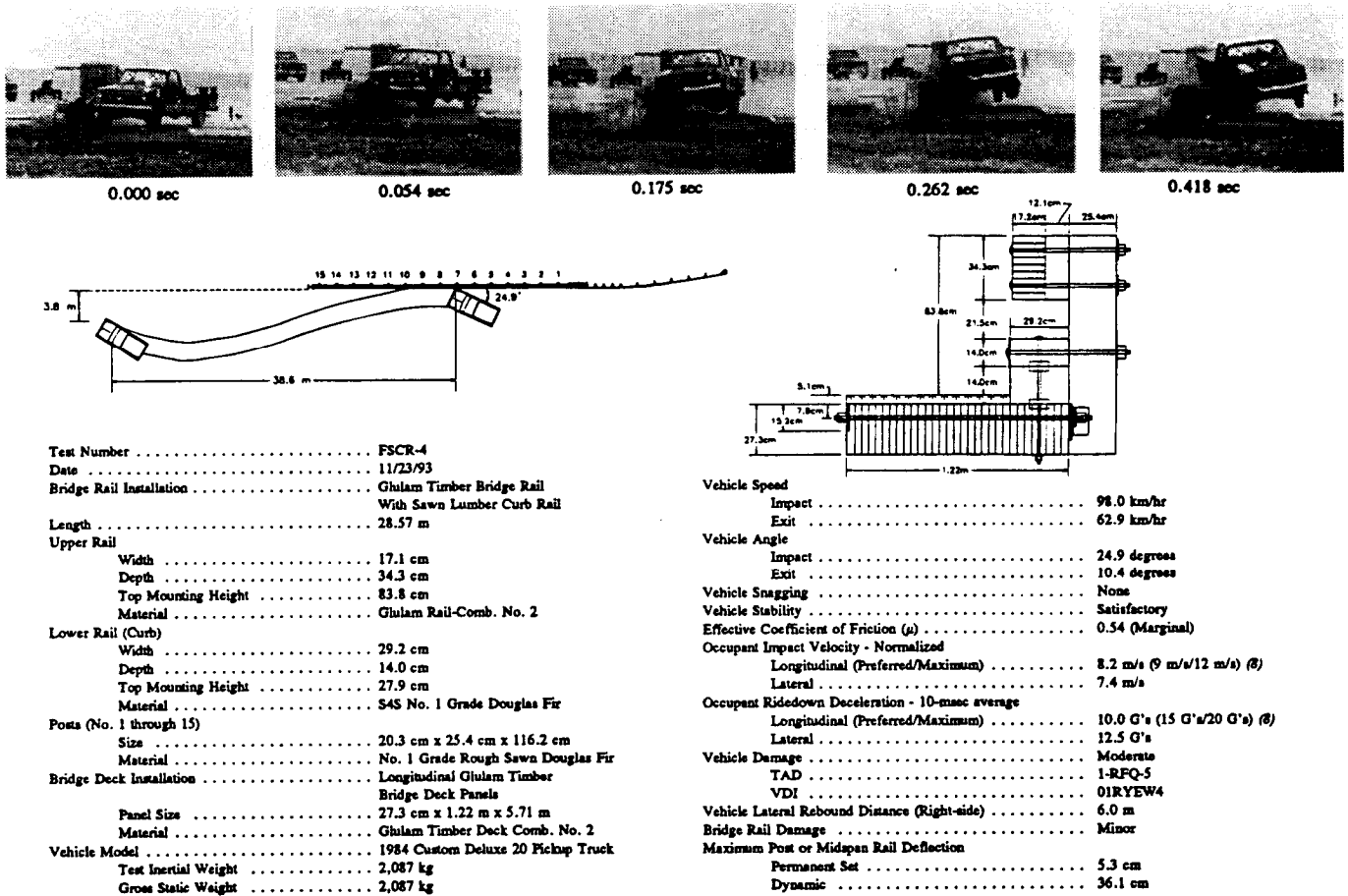


FIGURE 8 Summary of test results and sequential photographs, Test FSCR-4.

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.

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