

Railing Systems for Use on Timber Deck Bridges

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

Bridge railing systems in the United States have historically been designed based on static load criteria given in the AASHTO *Standard Specifications for Highway Bridges*. In the past decade, full-scale vehicle crash testing has been recognized as a more appropriate and reliable method of evaluating bridge railing acceptability. In 1989, AASHTO published the *Guide Specifications for Bridge Railings*, which gave the recommendations and procedures to evaluate bridge rails by full-scale vehicle crash testing. In 1993, the National Cooperative Highway Research Program (NCHRP) published *Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features*, which provided criteria for evaluating longitudinal barriers. Based on these specifications, a cooperative research program was initiated between the University of Nebraska-Lincoln and the Forest Products Laboratory, and later the FHWA, to develop and crash test 11 bridge rails for wood deck bridges. The research that resulted in successful development and testing of 11 bridge railing systems for longitudinally and transversely laminated wood bridge decks in accordance with AASHTO Performance Level 1 and 2 (PL-1 and PL-2) requirements and Test Levels 1, 2, and 4 (TL-1, TL-2, and TL-4) requirements of *NCHRP Report 350* are described here.

The primary purpose of a bridge railing is to safely contain errant vehicles crossing a bridge. To meet this objective, railings must be designed to withstand the force of an impacting vehicle without endangering the occupants in the vehicle. In designing railing systems for highway bridges, engineers traditionally have assumed that vehicle impact forces can be approximated by equivalent static loads that are applied to railing elements. Although rail loads are actually dynamic, the equivalent static load method has been used for many years as a simplified approach to standardized railing design. Until recently, AASHTO *Standard Specifications for Highway Bridges (1)* required that rail posts be designed to resist an outward transverse static load of 44.5 kN. A portion of this load was also applied to posts in the inward transverse, longitudinal, and vertical directions and to the rail elements. These requirements were identical for all bridges regardless of bridge geometry or traffic conditions. Thus, a railing for a single-lane bridge located on a low-volume road was required to meet the same loading requirements as a railing for a bridge located on a major highway.

Despite the widespread use of design requirements based primarily on static load criteria, the need for more appropriate full-scale vehicle crash test criteria has long been recognized. The first U.S. guidelines for full-scale vehicle crash testing were published in 1962 (2). This one-page document provided basic guidelines for the test vehicle

mass, approach speed, and impact angle and provided a degree of uniformity to the traffic barrier research in progress at the time. Through subsequent use of this document, the need for more comprehensive guidelines became apparent, and several reports were published during the 1970s through the NCHRP. In 1981, NCHRP published *Report 230, Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances (3)*. This comprehensive report provided recommendations relative to crash testing and evaluation of longitudinal barriers and served as the basis for future bridge rail crash-testing requirements.

Although crash test criteria have been available for many years, the requirement to implement crash testing as a means of evaluating bridge railings in the United States has been jurisdiction dependent. Some states implemented extensive bridge rail crash-testing programs, and others continued to use exclusively the static load design method. The first recognition of full-scale crash testing in a national bridge specification came in 1989 when AASHTO published *Guide Specifications for Bridge Railings (4)*. This specification presents recommendations for the development, testing, and use of crash-tested bridge railings and refers extensively to *NCHRP 230* for crash-testing procedures and requirements.

A primary concept of the AASHTO *Guide Specifications* was that bridge railing performance needs differ greatly from site to site, and railing designs and costs should match site needs. Thus, recommended requirements for rail testing were based on three performance levels: performance level 1 (PL-1), performance level 2 (PL-2), and performance level 3 (PL-3). The PL-1 requirements represent the "weakest" system, and the PL-3 the "strongest" system. The relationship between the railing performance level and requirements for a specific bridge depend on a number of factors, such as type of roadway, design speed, average daily traffic, and percentage of trucks in the traffic mix.

The recently published *NCHRP Report 350: Recommended Procedure for the Safety Performance Evaluation of Highway Features (5)*, provides for six test levels for evaluating longitudinal barriers: test level 1 (TL-1) through test level 6 (TL-6). Although this document does not include objective criteria for relating a test level to a specific roadway type, the lower test levels are generally intended for use on lower-service-level roadways and certain types of work zones and the higher test levels are intended for use on higher-service-level roadways.

In 1994, AASHTO published the *LRFD Bridge Design Specifications (6)* as an update to the *Standard Specifications for Highway Bridges (1)* and the *Guide Specifications for Bridge Railings (4)*. For crash testing bridge railings, three performance levels were provided similar to those in the *Guide Specifications for Bridge Railings (4)*. Guidelines for crash testing bridge railings followed procedures provided in both the *AASHTO Guide Specifications* and *NCHRP Report 350*. Yield line and inelastic analysis and design

R. K. Faller, Midwest Roadside Safety Facility, University of Nebraska-Lincoln, 1901 Y Street, Building C, Lincoln, NE 68588-0601. M. A. Ritter, Forest Products Laboratory, U.S. Department of Agriculture-Forest Service, One Gifford Pinchot Drive, Madison, WI 53705. B. T. Rosson, Civil Engineering Department, University of Nebraska-Lincoln, W348 Nebraska Hall, Lincoln, NE 68588-0531. S. R. Duwadi, Turner-Fairbank Highway Research Center, Federal Highway Administration, 6300 Georgetown Pike, McLean, VA 22101-2296.

procedures, as originally developed by Hirsch (7), were also provided for bridge railings as a replacement to the 44.5-kN equivalent static load design procedures.

Emphasis on the use of crash-tested rails for new federally funded projects has significantly increased the role of full-scale crash testing as a means of evaluating railing performance. Recently, the FHWA officially adopted *NCHRP 350* as a replacement for *NCHRP 230* and has strongly suggested that AASHTO also adopt the test level definitions contained in *NCHRP 350*, thus making crash-tested railings mandatory for most bridges. Most highways on which wood bridges are installed will require railings that meet the AASHTO PL-1 or PL-2 requirements or the *NCHRP 350* TL-1 through TL-4 requirements. A railing that meets either PL-3, TL-5, or TL-6 requirements currently has a very limited application for wood bridges because of the high traffic volumes and speeds associated with these levels.

As of August 1986, 22 bridge rails had been successfully crash tested in accordance with the guidelines specified in *NCHRP 230* and approved for use on federal-aid projects by the FHWA (8). By August 1990, 25 additional bridge rails had been successfully crash tested in accordance with the requirements of the *AASHTO Guide Specifications* and also approved by the FHWA for use on federal-aid projects (9). Of these crash-tested railings, 46 were for concrete bridge decks and only 1 was for a wood deck (10). For wood bridges to be viable and competitive with other bridges in the future, a range of crash-tested bridge railings for different wood bridge types was required. Based on this need, national emphasis was placed on developing a limited number of crash-tested railing systems for wood bridges.

BACKGROUND

To meet the need for crashworthy railings for wood bridges, the U.S. Department of Agriculture Forest Service, Forest Products Laboratory, in cooperation with the Midwest Roadside Safety Facility (MwRSF) of the University of Nebraska-Lincoln, the FHWA, and the wood products industry initiated a program to develop crash-tested bridge rails for both longitudinal wood decks (Phase I) and transverse wood decks (Phase II). Simultaneously with the Phase I research program at MwRSF, researchers at West Virginia University conducted a research effort to develop three AASHTO PL-1 railing systems for transverse wood decks (11).

Longitudinal Wood Decks

The program objectives for Phase I were to develop a total of nine crashworthy rails: three to meet AASHTO PL-1, one to meet AASHTO PL-2, three to meet *NCHRP 350* TL-1, one to meet *NCHRP 350* TL-4, and one intended for roadway applications with very low traffic volumes. The scope of Phase I was limited to railings for longitudinal wood decks, 252 mm or greater in thickness, and constructed of glued-laminated (glulam) timber, spike-laminated lumber, or stress-laminated lumber. In each system, the lumber laminations are placed edgewise and oriented with the lumber length parallel to the direction of traffic. A brief description of each longitudinal deck bridge type is provided in *Timber Bridges: Design, Construction, Inspection, and Maintenance* (12).

Longitudinal glulam timber decks are constructed of panels that consist of individual lumber laminations glued together with waterproof structural adhesives. The panels are 1.07 to 1.38 m wide and

effectively function as a large, solid block of wood. To form the bridge deck, panels are placed side by side and are interconnected by transverse distributor beams bolted to the deck underside at intervals of 2.4 m or less. These distributor beams are designed to transfer vertical loads between adjacent panels but they are not designed to resist lateral loads.

Spike-laminated decks are constructed of nominal 102-mm-thick sawed lumber laminations. The individual laminations are interconnected with spikes that are typically 8 or 9.5 mm in diameter and 356 to 406 mm long. The decks are commonly manufactured in panels that are 1.5 to 2.1 m wide and are interconnected with transverse distributor beams in a manner similar to longitudinal glulam timber decks.

Stress-laminated decks are constructed of sawed lumber laminations that are typically 51 to 102 mm in nominal thickness. The laminations are stressed together with high-strength steel bars that are placed through holes drilled through the center of the wide faces of the laminations. When tensioned, the bars create compression between the laminations, and the entire deck effectively acts as a solid, orthotropic wood plate.

Transverse Wood Decks

For Phase II, the program objectives were to develop four additional railing systems: two to meet *NCHRP 350* TL-4 and two to meet *NCHRP 350* TL-2. The scope of Phase II was limited to railings for transverse wood decks, 130 mm or greater in thickness, and constructed of glulam timber. For these systems, the lumber laminations are placed edgewise and oriented with the lumber length perpendicular to the direction of traffic.

Transverse glulam timber decks are constructed of panels that consist of individual lumber laminations glued together with waterproof structural adhesives. The panels are typically 1.22 m wide and effectively act as a thin plate. To form the bridge deck, panels are placed side by side and are supported by longitudinal glulam or steel beams. These longitudinal beams are designed to carry the vertical loads and are braced by either glulam or steel diaphragms to provide lateral stiffness to the bridge structure.

TEST REQUIREMENTS AND EVALUATION CRITERIA

The test requirements and evaluation criteria for this project followed procedures defined in the *AASHTO Guide Specifications* (including applicable references to *NCHRP 230*) and the *NCHRP 350* criteria. These procedures establish a uniform methodology for testing and evaluating railings so that the safety performance of different railing designs, tested and evaluated by different agencies, can be compared. It is impractical and impossible to test all railings for all possible vehicle and impact conditions. Therefore, the procedures specify a limited number of tests under severe vehicle impact conditions and a set of evaluation criteria against which test results may be evaluated.

Test Requirements

Vehicle impact requirements for rail crash testing depend on the railing performance level/test level and are specified as requirements for vehicle type and weight, impact speed, and impact angle relative to

the longitudinal rail axis. Testing for PL-1, TL-1, and TL-2 requires two vehicle impact tests and testing for PL-2 and TL-4 requires three vehicle impact tests. A summary of the requirements for PL-1, PL-2, TL-1, and TL-4 is presented in Table 1. In some cases, all tests for a given level may not be required if a railing with similar geometry and strength was previously tested and found to be satisfactory.

In addition to vehicle impact requirements, the *AASHTO Guide Specifications* and the *NCHRP 350* criteria also specify requirements for data acquisition and construction of the bridge railing. Requirements for data acquisition are referenced to *NCHRP 230* and *NCHRP 350* and include specific data collection parameters and techniques that must be completed before, during, and after the crash test. Construction requirements specify that the bridge rail be designed, constructed, erected, and tested in a manner representative of actual installations. To properly assess the performance of most bridge rails, they must also be evaluated as a system in combination with the bridge superstructure for which it is intended. This is very important when rails for wood bridges are being considered, because the attachment of the rail to the bridge deck and the ability of the wood superstructure to resist applied rail loads may often be the controlling parameters.

Evaluation Criteria

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

DEVELOPMENT PHASE

Longitudinal Panels

Based on a fundamental understanding of the performance characteristics of each deck type, development work was initiated to formulate a methodology for the railing tests. From the standpoint of economics and time, it was considered impractical to develop and test different rail systems for each longitudinal deck type. Rather, a more feasible approach was undertaken to develop several railing systems that could be adapted to each of the three longitudinal deck types with little or no modifications. To accomplish this, it was determined that railing development and testing should utilize the weakest deck type for resisting lateral impact forces. This conclusion was based on the premise that if successful tests could be completed on the weakest deck, the railing could be adapted to the stronger longitudinal wood decks without adversely affecting performance.

In assessing the potential resistance of each longitudinal deck type to transverse railing impact forces, consideration was given to the strength of the wood and mechanical reinforcement. Of primary concern was loading that could introduce tension perpendicular to grain stress in the wood deck.

Of the three longitudinal deck types, the stress-laminated deck was considered the strongest for transverse railing loads, because the high-strength steel bars are continuous across the deck width. Loads developed at vehicle impact can be effectively distributed across the deck by the bars, making the entire deck width effective in resisting the applied loads.

The spike-laminated deck was considered to be of intermediate strength. Because rail loads are applied transverse to the panel length, the loads are resisted by the spikes in withdrawal. Because of this, tension perpendicular to the grain in the lumber laminations is not a concern; however, the spikes could be pulled from the deck, resulting in longitudinal separations between the laminations, and additional reinforcement could be required.

The glulam timber deck was considered to be the weakest in resisting transverse railing loads, because the glulam timber panels act as solid pieces of wood, and loads applied transverse to the panel length are most likely to introduce tension perpendicular to grain

TABLE 1 Vehicle Impact Requirements for AASHTO PL-1 and PL-2 and NCHRP 350 TL-1, TL-2, and TL-4 Bridge Railings

| AASHTO Performance Level (4) | Impact Conditions | | |
|------------------------------|---------------------|-------------------------|-------------------------------------|
| | Small Car (816 kg) | Pickup Truck (2,449 kg) | Medium Single-Unit Truck (8,165 kg) |
| 1 | 80.5 km/h 20 deg | 72.4 km/h 20 deg | |
| 2 | 96.6 km/h 20 deg | 96.6 km/h 20 deg | 80.5 km/h 15 deg |
| NCHRP 350 Test Level (5) | Impact Conditions | | |
| | Small Car (820 kg) | Pickup Truck (2,000 kg) | Single-Unit Van Truck (8,000 kg) |
| 1 | 50 km/h 20 deg | 50 km/h 25 deg | |
| 2 | 70 km/h 20 deg | 70 km/h 25 deg | |
| 4 | 100 km/h 20 deg | 100 km/h 25 deg | 80 km/h 15 deg |

and failure in the upper panel section. Mechanical reinforcement was considered necessary for longitudinal glulam timber decks to resist railing loads without damage. Thus, the glulam timber deck was selected for full-scale crash testing (Phase I).

Transverse Panels

Highway bridges using transverse timber decks and those requiring crash-tested railing systems are most commonly constructed with glulam timber deck panels. These panels are typically 1.22 m wide and 127 to 171 mm thick. Given the panel orientation perpendicular to traffic, railing loads primarily introduce tension and bending in the panels parallel to the wood grain. Unlike the case with the longitudinal glulam timber decks, tension perpendicular to the wood grain is not a primary design consideration.

Bridge Rail Design

The primary emphasis of the railing design process was to develop rails that would meet the requirements of the *AASHTO Guide Specifications* and *NCHRP 350*. Additionally, it was determined that consideration should be given to (a) the extent of probable damage to the structure after vehicle impact and the difficulty and cost of required repairs; (b) the adaptability of the railing to different wood deck types; (c) the rail system cost to the user, including material, fabrication, and construction; (d) the ease of railing construction and maintenance; and (e) aesthetics.

The conclusion of the development phase involved the design of several railing systems and preparation of plans and specifications for testing. The selection and design of these final systems were based on a review of other railings that had been successfully crash tested as well as those that are currently used on wood bridges but had not been crash tested. To the extent possible, feasible designs were evaluated by BARRIER VII computer simulation modeling (13). Although several proven computer models were used, it was sometimes difficult to adapt the programs for wood components because the behavior and properties of the wood systems at ultimate loading were unknown. Data collected during the crash testing were used to refine input parameters and to more accurately predict railing performance in later tests.

TEST METHODOLOGY

Testing of all bridge rails was conducted at MwRSF's outdoor test site in Lincoln, Nebraska. The site is located at an airport and was formerly a taxiway and parking area for military aircraft. It includes approximately 11 ha of concrete pavement and 1.6 ha of soil surface. To perform all the rail testing, two different test bridges were constructed.

For the longitudinal deck systems, a test bridge was constructed that measured approximately 2.44 m wide and 28.58 m long in five simply supported spans measuring 5.72 m each. The deck was constructed of 273-mm-thick and 1.22-m-wide glulam timber panels. The glulam timber for the deck was combination no. 2 Douglas fir given in the *AASHTO Standard Specifications for Highway Bridges* (1) and was treated with pentachlorophenol in heavy oil in accordance with American Wood Preservers Association (AWPA) Standard C14 (14). Two glulam timber panels were placed side by side to achieve the 2.44-m width, and transverse distributor beams were

attached to the deck underside according to AASHTO requirements (1). The test bridge was positioned on concrete supports that were placed in excavations so that the top of the test bridge was level with the concrete surface at the site.

For the transverse deck systems, a second test bridge was constructed that measured approximately 3.96 m wide and 36.58 m long in three simply supported spans measuring approximately 12.19 m each. The deck was constructed of 130-mm-thick and 1.22-m-wide glulam timber panels. The glulam timber for the deck was combination no. 47 Southern Yellow pine given in the *AASHTO LRFD Bridge Design Specification* (6) and it was also treated according to AWPA Standard C14 (14). Thirty glulam timber panels were placed side by side to achieve the 36.58-m length and they were attached to the longitudinal glulam beams with standard aluminum deck brackets. This test bridge was positioned on concrete supports that were placed in a 2.13-m deep excavated test pit. The concrete supports were placed so that the top of the test bridge was 51 mm below the concrete surface to allow for placement of the bridge deck wearing surface.

Vehicle propulsion and guidance were provided by steel cable configurations. For propulsion, a reverse cable tow with a 1:2 mechanical advantage was used. A cable was attached to the front of the vehicle, routed through a series of pulleys, and connected to a tow vehicle that traveled in a direction opposite that of the test vehicle. The unoccupied test vehicle was then pulled by the tow vehicle and released from the tow cable a prescribed distance before impact. A vehicle guidance system developed by Hinch et al. was used to steer the test vehicle (15). With this system, the left-front wheel hub is attached to a tensioned steel cable that maintains the vehicle's direction along a designated straight path. Before impact, the guidance connection is sheared off and the vehicle separates from the guidance cable.

Data acquisition parameters and techniques for the crash-testing program were based on requirements of the *AASHTO Guide Specifications* and *NCHRP 350* and followed three testing phases: pretest, test, and posttest. In the pretest phase, the as-built bridge rail and vehicle were documented by photography and drawings that indicated the applicable configuration, dimensions, and vehicle weight. During the test phase, data about the vehicle impact speed, impact angle, trajectory, and accelerations were collected primarily through the use of high-speed motion picture photography and accelerometers mounted on the vehicle. In the posttest phase, the condition of the railing, bridge superstructure, and vehicle were documented by photography and standardized damage assessment methods, including the traffic accident data scale (16) and the vehicle damage index (17). Additional instrumentation was placed on some railings to assess vehicle impact forces transmitted to the bridge rail and superstructure (18,19).

CRASH-TESTED RAILINGS FOR LONGITUDINAL DECKS

As a result of the Phase I development and testing program, nine bridge railings were successfully developed and tested for longitudinal wood decks. Three of the railings were tested at PL-1, one was tested at PL-2, three were tested at TL-1, one was tested at TL-4, and one was tested primarily for low-volume forest road applications at impact conditions less than TL-1. Each railing was tested on the glulam timber deck and is adaptable to the spike-laminated and stress-laminated decks. All the PL-1, PL-2, and TL-4 designs used high-strength steel bars through a portion of the bridge deck to act as reinforcement in distributing railing loads without damage to the

bridge. Glulam timber for the rail members was combination no. 2 Douglas fir as given in the *AASHTO Standard Specifications for Highway Bridges (I)*, treated with pentachlorophenol in heavy oil to AWWA Standard C14 requirements (14). Sawed lumber for posts, curbs, scuppers, and spacer blocks was no. 1 Douglas fir (I), treated with creosote to AWWA Standard C14 requirements (14).

A detailed discussion of the testing and results for each railing system is beyond the scope of this paper and, for most of the railing systems, is presented in detail in previous publications (18,20-24). Overall, no significant damage to the test bridge was evident from any of the vehicle impact tests. For the railing systems with glulam timber rails, damage to the railing was primarily gouging and scraping resulting from the vehicle impact. All glulam timber railing remained intact and serviceable after the tests, and replacement of

the railing was not considered necessary. For the steel thrie-beam railings, there was permanent deformation in the rail and post in the vicinity of the impact location. This would necessitate replacement of specific railing and post members, but, considering the severity of the impact, damage was relatively minor. A brief description of each railing design follows.

PL-1 Railings

The three tested PL-1 railings included a glulam timber rail with curb, a glulam timber rail without curb, and a steel rail. Photographs and drawings of the PL-1 railings are presented in Figures 1 and 2, respectively.

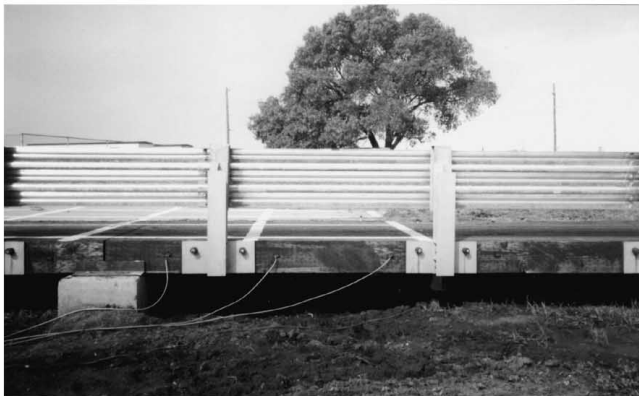
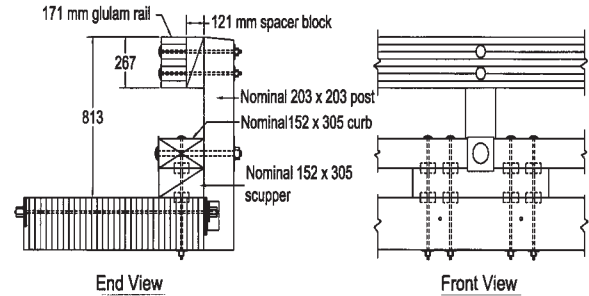
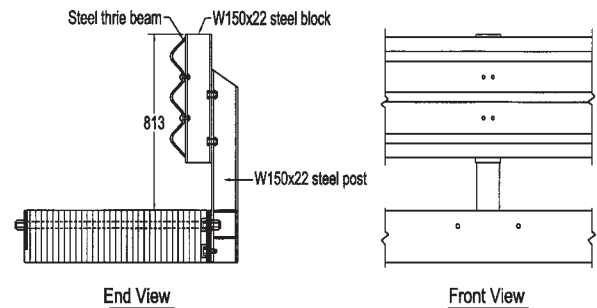


FIGURE 1 Bridge railings successfully crash tested to AASHTO PL-1 (longitudinal deck).

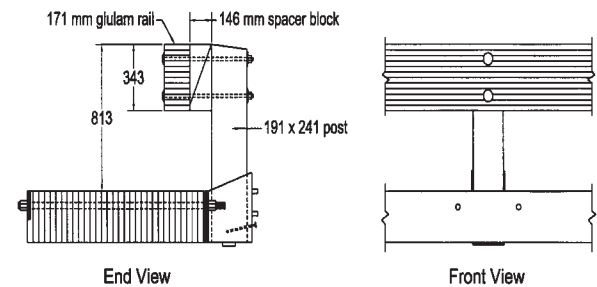
Glulam Timber Rail with Curb



Steel Rail



Glulam Timber Rail without Curb



*Dimensions in mm.

FIGURE 2 Details of bridge railings developed according to AASHTO PL-1 (longitudinal deck).

The glulam timber rail with curb consisted of a single glulam timber rail mounted on sawed lumber posts spaced 1905 mm on center. The post was connected with a single bolt to a lumber curb that was supported by scupper blocks. The curb and scupper blocks were connected to the bridge deck with bolts and timber connectors.

The glulam timber rail without curb consisted of a single glulam timber rail mounted on sawed lumber posts spaced 1905 mm on center. The lower portion of the post was placed in a steel box that was attached to the bridge deck with high-strength steel bars.

The steel rail consisted of a 10-gauge steel thrie-beam rail mounted to steel wide-flange posts spaced 1905 mm on center. The lower end of the post was bolted to a steel plate that was connected to the bridge deck with high-strength steel bars.

PL-2 Railing

The PL-2 railing included a steel rail and steel channel section, as indicated in Figure 3(a). The steel rail was a modified version of that tested at PL-1. Post spacing was 1905 mm on center. Minor changes in the railing geometry and the addition of a steel channel section above the rail were necessary to resist the increased loads at PL-2.

TL-4 Railing

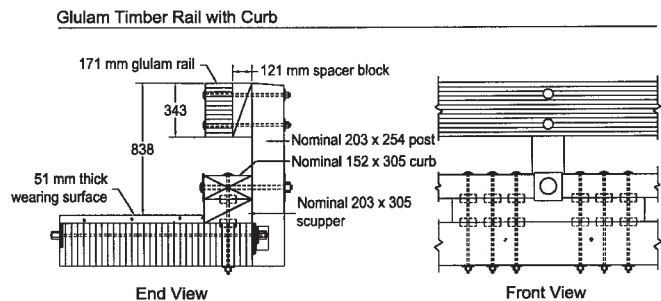
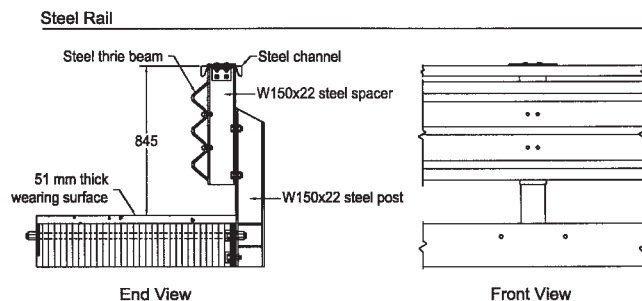
The TL-4 railing included a glulam timber rail with curb, as indicated in Figure 3(b). The glulam timber rail with curb consisted of a single glulam timber rail mounted on a sawed lumber post and was a modification of the curb system tested at PL-1. Post spacing was 1905 mm on center. Because of the greater loads at TL-4, rail and post sizes were increased, and bolts and timber connectors attaching the curb and scupper to the bridge deck were increased.

TL-1 Railings

The three tested TL-1 railings included a flexible steel rail, a rigid steel rail, and a curb-type timber rail. Photographs and drawings of the TL-1 railings are presented in Figures 4 and 5, respectively.

The flexible steel rail consisted of a 12-gauge W-beam rail mounted to breakaway sawed lumber posts spaced 1905 mm on center. The lower end of the post was placed between two steel angles that were connected to the vertical edge of the bridge deck with lag screws.

The rigid steel rail consisted of a 12-gauge W-beam rail mounted to steel wide-flange posts spaced 1905 mm on center. The post



*Dimensions in mm.

(a)

(b)

FIGURE 3 (a) Steel thrie-beam bridge railing successfully crash tested to AASHTO PL-2 (longitudinal deck); (b) Glulam timber bridge railing successfully crash tested to NCHRP 350 TL-4 (longitudinal deck).



FIGURE 4 Bridge railings successfully crash tested to NCHRP 350 TL-1 (longitudinal deck).

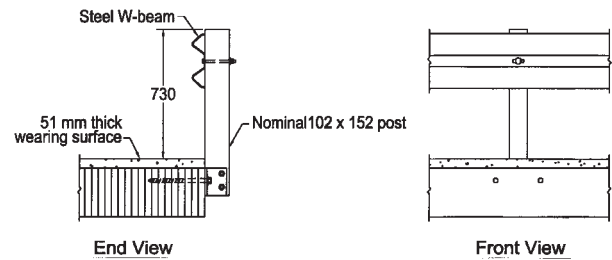
was bolted to a steel plate, which was bolted to the bridge deck surface.

The low-height, curb-type timber rail was constructed with a glulam timber rail and supported with scupper blocks. The curb and scupper blocks, spaced 3048 mm on center, were connected to the bridge deck with bolts and timber connectors.

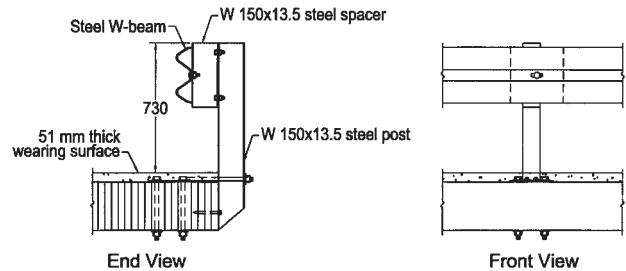
Railing for Very Low Volume Roads

The railing developed for impact conditions less than TL-1, such as for very low volume roadway applications, included a low-height,

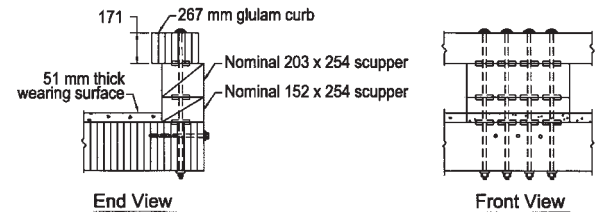
Flexible Steel Rail



Semi-Rigid Steel Rail



Curb-type Rail



*Dimensions in mm.

FIGURE 5 Details of bridge railings developed according to NCHRP 350 TL-1 (longitudinal deck).

timber curb rail, as indicated in Figure 6. Three geometries were considered for the curb rail: a square shape, a rectangular shape, and a trapezoidal shape. The curb rail was constructed with sawed lumber and supported with scupper blocks. The curb and scupper blocks, spaced 2896 mm on center, were connected to the bridge deck with bolts.

CRASH TESTED RAILINGS FOR TRANSVERSE DECKS

Phase II of the research program is ongoing. Thus far, this phase has resulted in two bridge railings that were successfully developed and tested for transverse wood decks. These two railings—one wood system and one steel system—were tested according to the TL-4 impact conditions. Two additional bridge railing systems are currently under development and are intended to meet the TL-2 performance criteria. The TL-4 railings were tested on the transverse

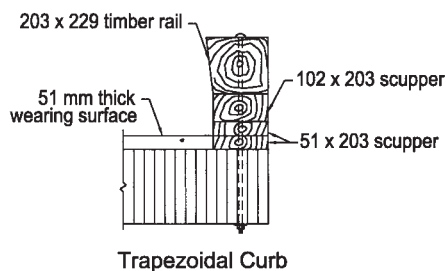
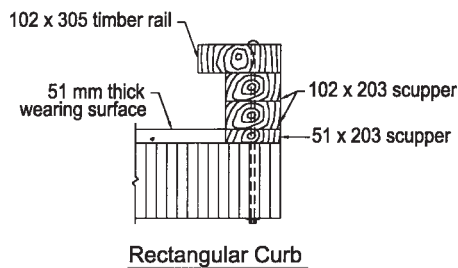
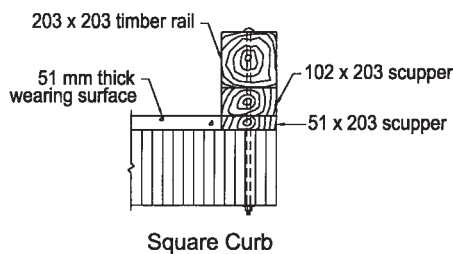
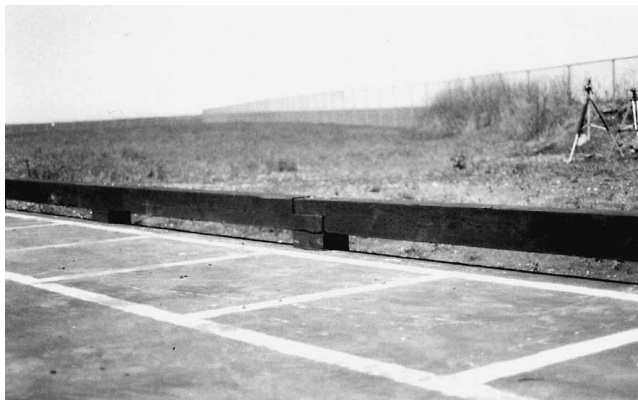


FIGURE 6 Low-volume railing successfully crash tested with a 0.75-ton (0.68 Mg) pickup truck at 15 mph (24.14 km/h) and 15° (longitudinal deck).

glulam timber deck. Both of the TL-4 designs used posts spaced 2438-mm on center. For the wood system, glulam timber for the upper rail and post members was combination no. 48 Southern Yellow pine as given in the *AASHTO LRFD Bridge Design Specifications* (6), treated with pentachlorophenol in heavy oil to AWPA Standard C14 requirements (14). Glulam timber for the curbs, scuppers, and spacer blocks was fabricated with combination no. 47 Southern Yellow pine as given by AASHTO (6) and treated in the same manner as described previously according to AWPA Standard C14 (14).

Test details and results for the two TL-4 railing systems are not included here and, for the wood railing system, are provided in detail in a previous publication (19). No significant damage to the test bridge was evident from any of the vehicle impact tests. For the railing system with glulam timber rails, damage to the railing was primarily gouging and scraping resulting from the vehicle impact. All glulam timber railings remained intact and serviceable after the tests and replacement of the railing was not considered necessary. For the steel thrie-beam railing, there was permanent deformation in the rail and post in the vicinity of the impact location. This would necessitate replacement of specific railing and post members, but damage was very minor considering the severity of the impact. A brief description of each railing design follows.

The two tested TL-4 railings included a glulam timber rail with curb and a steel rail. Photographs and drawings of the TL-4 railings are presented in Figure 7.

The first TL-4 railing consisted of a glulam timber rail with curb, as shown in Figure 7(a). This system was configured similarly to the PL-1 and TL-4 glulam timber rail with curb systems developed for longitudinal decks; however, for this system, all wood components were fabricated from glulam timber. In addition, all structural members as well as the steel hardware were resized to account for the increased post spacing from 1905 to 2438 mm. The new post spacing was selected to optimize the design and significantly improve the constructibility of the railing system, which was based on 1219-mm-wide deck panels.

The steel rail consisted of a 10-gauge steel thrie-beam mounted to a steel wide-flange post, as indicated in Figure 7(b). The lower end of the post was bolted to two steel plates that were connected to the top and bottom surfaces of the bridge deck with vertical bolts. As a result of the increased post spacing, the channel rail used in the PL-2 system for longitudinal decks was replaced with a steel structural tube. This change was made to provide increased load distribution and resistance to lateral buckling of the tube.

CONCLUDING REMARKS

This program clearly demonstrates that crashworthy railing systems are feasible for both longitudinal and transverse wood decks. Even at high-impact conditions required by AASHTO PL-2 and *NCHRP 350 TL-4*, the railing systems performed well with no significant damage to the bridge superstructure. With the development of crashworthy railing systems, a significant barrier to the use of longitudinal and transverse wood deck bridges has been overcome.

At the onset of this research program, only one crash-tested bridge railing was available for use on wood deck bridges. Over the past 10 years, this cooperative research program has resulted in the development of 11 crash-tested bridge rails for use on both longitudinally and transversely laminated timber deck bridges.

ACKNOWLEDGMENTS

The authors wish to thank the following organizations, which have contributed to the overall success of this project: the American Institute of Timber Construction, Englewood, CO; the Forest Products Laboratory, Madison, WI; the FHWA, Washington, D.C.; Alamco Wood Products, Inc., Albert Lea, MN; Western Wood Structures, Inc., Tualatin, OR; Laminated Concepts, Elmira, NY;

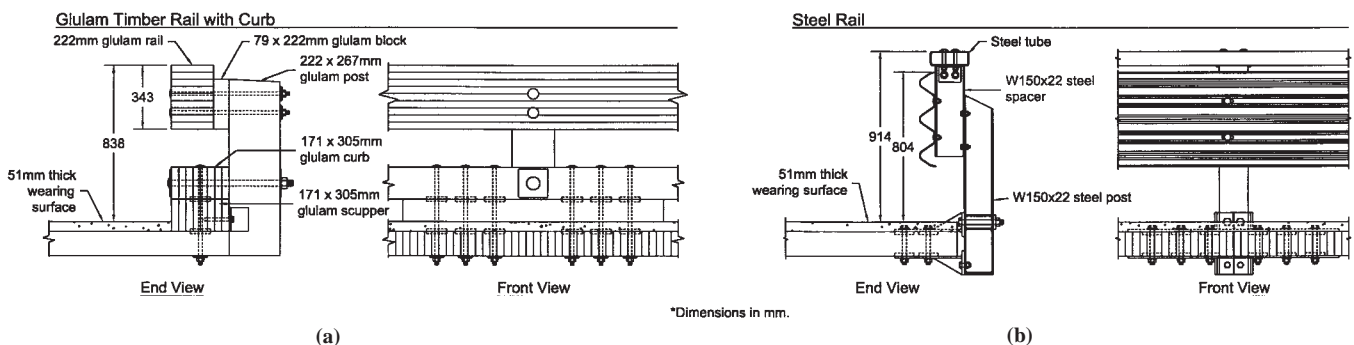


FIGURE 7 (a) Glulam timber bridge railing successfully crash tested to NCHRP 350 TL-4 (transverse deck); (b) steel thrie-beam bridge railing successfully crash tested to NCHRP 350 TL-4 (transverse deck).

Wheeler Consolidated, Inc., St. Louis Park, MN; Hughes Brothers, Seward, NE; Buffalo Specialty Products—Timber Division, Sunbright, TN; Midwest Transportation Center, Ames, IA; Office of Sponsored Programs and Center for Infrastructure Research, University of Nebraska-Lincoln, Lincoln, NE. Finally, a special thanks is given to all the MwRSF personnel for constructing the bridge structures and barriers and for conducting the crash tests.

REFERENCES

1. *Standard Specifications for Highway Bridges*. AASHTO, Washington, D.C., 1989.
2. Full-Scale Testing Procedures for Guardrails and Guide Posts. *Highway Research Circular No. 482*. HRB, National Research Council, Washington, D.C., 1962.
3. Michie, J. D. *NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances*. TRB, National Research Council, Washington, D.C., 1981.
4. *Guide Specifications for Bridge Railings*. AASHTO, Washington, D.C., 1989.
5. Ross, H. E., Jr., D. L. Sicking, R. A. Zimmer, and J. D. Michie. *NCHRP Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features*. TRB, National Research Council, Washington, D.C., 1993.
6. *AASHTO LRFD Bridge Design Specifications*, 1st ed. AASHTO, Washington, D.C., 1994.
7. Hirsch, T. J. *Analytical Evaluation of Texas Bridge Rails to Contain Buses and Trucks*. Report No. FHWA/TX78-230-2. Submitted to Texas State Department of Highways and Public Transportation, Texas Transportation Institute, Texas A&M University, August 1978.
8. *Memorandum on Bridge Rails*. File Designation HNG-10/HHS-10, August 28, 1986. FHWA, Washington, D.C., 1986.
9. *Memorandum on Crash Tested Bridge Railings*. File Designation HNG-14, August 13, 1990. FHWA, Washington, D.C., 1990.
10. Hancock, K. L., A. G. Hansen, and J. B. Mayer. *Aesthetic Bridge Rails, Transitions, and Terminals for Park Roads and Parkways*. Report No. FHWA-RD-90-052. Submitted to the Office of Safety and Traffic Operations R&D, FHWA, performed by The Scientex Corp., May 1990.
11. Raju, P. R., H. V. S GangaRao, S. R. Duwadi, and H. K. Thippeswamy. Development and Testing of Timber Bridge and Transition Rails for Transverse Glued-Laminated Bridge Decks. In *Transportation Research Record 1460*, TRB, National Research Council, Washington, D.C., 1994, pp. 8–18.
12. Ritter, M. A. *Timber Bridges: Design, Construction, Inspection, and Maintenance*. EM-7700-8. USDA Forest Service, Washington, D.C., 1990.
13. Powell, G. H. *BARRIER VII: A Computer Program for Evaluation of Automobile Barrier Systems*. Report FHWA RD-73-51. FHWA, U.S. Department of Transportation, April 1973.
14. *AWPA Book of Standards*. American Wood Preservers Association, Woodstock, Md., 1992.
15. Hinch, J., T. L. Yang, and R. Owings. *Guidance Systems for Vehicle Testings*. ENSCO Inc., Springfield, Va., 1986.
16. *Vehicle Damage Scale for Traffic Accident Investigators*, 2nd ed. Technical Bulletin No. 1, Traffic Accident Data (TAD) Project, National Safety Council, Chicago, Ill., 1971.
17. *Collision Deformation Classification—Recommended Practice J224 March 1980*. Handbook, Vol. 4, Society of Automotive Engineers, Warrendale, Penn., 1985.

18. Faller, R. K., M. A. Ritter, J. C. Holloway, B. G. Pfeifer, B. T. Rosson. Performance Level 1 Bridge Railings for Timber Decks. In *Transportation Research Record 1419*, TRB, National Research Council, Washington, D.C., 1993, pp. 21–34.
19. Fowler, M. D. *Design and Testing of a Test Level 4 Bridge Railing for Transverse Glulam Timber Deck Bridges*. M.S. thesis. University of Nebraska-Lincoln, May 1997.
20. Ritter, M. A., and R. K. Faller. *Crashworthy Bridge Railings for Longitudinal Wood Decks*. 1994 Pacific Timber Engineering Conference, Gold Coast, Australia, July 11–15, 1994.
21. Rosson, B. T., R. K. Faller, and M. A. Ritter. Performance Level 2 and Test Level 4 Bridge Railings for Timber Decks. In *Transportation Research Record 1500*, TRB, National Research Council, Washington, D.C., 1995, pp. 102–111.
22. Faller, R. K., B. T. Rosson, M. A. Ritter, and D. L. Sicking. Design and Evaluation of Two Low-Volume Bridge Railings. *Sixth International Conference on Low-Volume Roads, Vol. 2, Conference Proceedings 6*, University of Minnesota, Minneapolis, Minn., June 25–29, 1995, TRB, National Research Council, Washington, D.C., 1995.
23. Ritter, M. A., R. K. Faller, and S. R. Duwadi. Crash-Tested Bridge Railings for Timber Bridges. *Fourth International Bridge Engineering Conference, Vol. 2, Conference Proceedings 7*, San Francisco, Calif., Aug. 28–30, 1995, TRB, National Research Council, Washington, D.C., 1995.
24. Ritter, M. A., P. D. H. Lee, R. K. Faller, B. T. Rosson, and S. R. Duwadi. *Plans for Crash Tested Bridge Railings for Longitudinal Wood Decks*. General Technical Report No. FPL-GTR-87. United States Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, Wis., Sept. 1995.

The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented here. The contents do not necessarily reflect the official views or policies of the U.S. Department of Agriculture, Forest Service, Forest Products Laboratory, or the FHWA. This report does not constitute a standard, specification, or regulation.

Publication of this paper sponsored by Committee on General Structures.