

Railing Systems for Longitudinal Timber Deck Bridges

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Abstract

Bridge railing systems in the United States have historically been designed based on static load criteria given in the American Association of State Highway and Transportation Officials (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 gives recommendations and procedures to evaluate railings 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 provides 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 Federal Highway Administration, to develop and crash test several bridge railings for longitudinal wood decks. This paper describes the successful development and testing of nine resulting railing systems in accordance with the AASHTO Performance Level 1 and 2 (PL-1 and PL-2) requirements, and the Test Level 1 and 4 (TL-1 and TL-4) requirements of *NCHRP Report 350*.

Keywords: Timber Bridges, Bridge Rail, Crash Testing, Roadside Safety, and Longitudinal Barrier.

Introduction

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. In designing railing systems for highway bridges, engineers have traditionally assumed that vehicle impact forces can be approximated by equivalent static loads that are applied to railing elements. 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, the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Highway Bridges* (1) required that rail posts be designed to resist an outward transverse static load of 44.5 kN (10,000 lb). 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 initial 1-page document provided basic guidelines for the test vehicle mass, approach speed, and impact angle and served to provide 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 National Cooperative Highway Research Program (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, while others continued to exclusively use static load design. 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* is 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 are 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 the 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 while 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 provided 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 procedures, as originally developed by Hirsch (7), were also provided for bridge railings as a replacement to the 44.5 kN (10,000 lb) 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 Federal Highway Administration (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 either 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 1990, 25 bridge rails had been successfully crash tested in accordance with the requirements of the *AASHTO Guide Specifications* and approved for use on Federal-aid projects by the FHWA (8). Of these crash-tested railings, 24 are for concrete bridge decks and one is for a wood deck. 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 railings for wood bridges.

Background

To meet the need for crashworthy railings for wood bridges, the USDA Forest Service, Forest Products Laboratory (FPL), 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 longitudinal wood decks. The program objectives 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 very low performance conditions. The scope of the project was limited to railings for longitudinal wood decks, 252 mm (10 in.) 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* (9).

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 (3.5 to 4.5 ft) 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 (8 ft) or less. These distributor beams are designed to transfer vertical loads between adjacent panels. They are not designed to resist lateral loads.

Spike-laminated decks are constructed of sawn lumber laminations 102 mm (4 in.) in nominal thickness. The individual laminations are interconnected with spikes that are typically 8 or 9.5 mm (0.3125 or 0.375 in.) in diameter and 356 to 406 mm (14 to 16 in.) long. The decks are commonly manufactured in panels that are 1.5 to 2.1 m (5 to 7 ft) wide and interconnected with transverse distributor beams in a manner similar to longitudinal glulam timber decks.

Stress-laminated decks are constructed of sawn lumber laminations that are typically 51 to 102 mm (2 to 4 in.) 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.

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 using 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 and TL-1 requires two vehicle impact tests while 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 are shown 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 considering rails for wood bridges because the attachment of the rail to the bridge deck and the ability of the wood superstructure to resist applied loads may often be the controlling parameters.

Evaluation Criteria

Evaluation criteria for full-scale crash testing is based on three appraisal areas: structural adequacy, occupant risk, and vehicle trajectory after the collision. Criteria for structural adequacy are intended to evaluate the ability of the railing to contain, redirect, or permit

Table 1 – Vehicle impact requirements for AASHTO PL-1 and PL-2 and NCHRP 350 TL-4 bridge railings.

AASHTO Performance Level (4)	Impact Conditions		
	Small Car (816 kg)	Pickup Truck (2,449 kg)	Single-Unit Van 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	
4	100 km/h 20 deg	100 km/h 25 deg	80 km/h 15 deg

controlled vehicle penetration in a predictable manner. Occupant risk evaluates the degree of hazard to occupants of the impacting vehicle. Vehicle trajectory after the collision is concerned with the path and final position of the impacting vehicle and the probable involvement of the impacting vehicle with other traffic. 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 post-impact maintenance requirements. The following evaluation criteria are summarized from the *AASHTO Guide Specifications* for PL-1 and PL-2 testing (similar evaluation criteria are provided in *NCHRP 350*):

1. The railing shall contain the vehicle; neither the vehicle nor its cargo shall penetrate or go over the installation. Controlled lateral deflection of the railing is acceptable.
2. Detached elements, fragments, or other debris from the railing shall not penetrate or show potential for penetrating the passenger compartment or present undue hazard to other traffic.
3. Integrity of the passenger compartment must be maintained with no intrusion and essentially no deformation.

4. The vehicle shall remain upright during and after collision.

5. The railing shall smoothly redirect the vehicle. A redirection is deemed smooth if the rear of the vehicle does not yaw more than 5 degrees away from the railing from time of impact until the vehicle separates from the railing.

6. The smoothness of the vehicle-railing interaction is further assessed by the effective coefficient of friction where = 0.0 to 0.25 is good, = 0.26 to 0.35 is fair, and > 0.36 is marginal. Requirements for computing are given in the *AASHTO Guide Specifications*.

7. The impact velocity of a hypothetical front-seat passenger against the vehicle interior, calculated from vehicle accelerations and 610-mm (2-ft) longitudinal and 305-mm (1-ft) lateral displacements, shall be less than 9.15 m/s (30 ft/s) in the longitudinal direction and 7.63 m/s (25 ft/s) in the lateral direction. In addition, the vehicle highest 10-ms average accelerations subsequent to the instant of hypothetical passenger impact should be less than 147 m/s² (483 ft/s²) in the longitudinal and lateral directions.

8. Vehicle exit angle from the barrier shall not be more than 12 degrees. Within 30.5m (100 ft) plus the length of the test vehicle from the point of initial impact with the railing, the railing side of the vehicle shall move no more than 6.1 m (20 ft) from the line of the traffic face of the railing.

Development Phase

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 without modifications that would result in reduced performance. To accomplish this, it was determined that railing development and testing should utilize the weakest deck type. This conclusion was based on the premise that if successful tests could be completed on the weakest deck, the railing could be adapted for use on stronger decks without adversely affecting performance.

In assessing the potential resistance of each 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 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. If rail loads are applied transverse to the panel length, the loads are resisted by the spikes in withdrawal. Because of this, tension perpendicular to 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 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 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 considered the weakest deck for transverse railing loads and was selected for full-scale crash testing. If bridge railings performed acceptably on the glulam timber system, it was rationalized that the railings could be adapted to other longitudinal wood bridge decks with no reduction in railing performance.

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 (1) extent of probable damage to the structure after vehicle impact and the difficulty and cost of required repairs; (2) adaptability of the railing to different wood deck types; (3) rail system cost to the user, including material, fabrication, and construction; (4) ease of railing construction and maintenance; and (5) 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 using computer simulation models. Although several proven computer models were used, it was 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 subsequent tests.

Test Methodology

Testing of all bridge rails was conducted at the Midwest Roadside Safety Facility 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 (27 acres) of concrete pavement and 1.6 ha (4 acres) of soil surface. To perform the rail testing, a test bridge was constructed that measured approximately 2.4 m (8 ft) wide and 28.6 m (93.75 ft) long, in five simply-supported spans measuring 5.72 m (18.75 ft) each. The deck was constructed of 273-mm (10.75 in.) thick glulam timber panels, 1.2 m (4 ft) wide. 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 (10). Two glulam timber panels were placed side by side to achieve the 2.4-m (8-ft) width, and transverse distributor beams were attached to the deck underside per AASHTO requirements (1). The test bridge was supported by concrete footings that were placed in excavations so that the top of the test bridge was level with the concrete surface at the site.

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 was connected to a tow vehicle that traveled in a direction opposite to the test vehicle. The unoccupied test vehicle was then pulled by the tow vehicle and released from the tow cable approximately 9.2 m (30 ft) prior to impact. A vehicle guidance system developed by Hinch was used to steer the test vehicle (11). Using 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. Approximately 9.2 m (30

ft) from 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 post-test. In the pretest phase, the as-built bridge rail and vehicle were documented using photography and drawings that indicated the applicable configuration, dimensions, and vehicle weight. During the test phase, data regarding 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 post-test phase, the condition of both the railing, bridge superstructure, and vehicle were documented using photography and standardized damage assessment methods, including the Traffic Accident Data Scale (12) and Vehicle Damage Index (13). Additional instrumentation was placed on some railings to assess vehicle impact forces transmitted to the bridge rail and superstructure (14).

Successfully Tested Railings

As a result of the 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 to 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 employed posts spaced 1.9 m (6.25 ft) on-center and utilized high strength steel bars through a portion of the bridge deck to act as reinforcement in distributing railing loads without damage to the bridge. Two of the three TL-1 designs employed posts spaced 1.9 m (6.25 ft) on-center, while the third TL-1 design was a curb railing configured with scuppers spaced 3.0 m (10 ft) on-center. The railing design for conditions less than TL-1 consisted of a curb railing with scuppers spaced 2.9 m (9.5 ft) on-center. Glulam timber for the rail members was Combination No. 2 Douglas Fir as given in the *AASHTO Standard Specifications for Highway Bridges* (1), treated with pentachlorophenol in heavy oil to AWWA Standard C14 requirements (10). Sawn lumber for posts, curbs, scuppers, and spacer blocks was No. 1 Douglas Fir (1), treated with creosote to AWWA Standard C14 requirements (10).

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 (14-18). 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 damage was relatively minor considering the severity of the impact. A brief description of each railing that met all test criteria 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 shown in Figures 1 and 2, respectively.

The glulam timber rail with curb consisted of a single glulam timber rail mounted on a sawn lumber post. 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 a sawn lumber post. 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 a steel wide flange post. 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 shown in Figure 3. The steel rail was a modified version of that tested at PL-1. 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-1 Railings

The three tested TL-1 railings included a flexible steel rail, a semi-rigid steel rail, and a curb-type timber rail.

Photographs and drawings of the TL-1 railings are shown in Figures 4 and 5, respectively.

The flexible steel rail consisted of a 12-gauge W-beam rail mounted to a breakaway sawn lumber post. 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 semi-rigid steel rail consisted of a 12-gauge W-beam rail mounted to a steel wide flange post. The post 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 were connected to the bridge deck with bolts and timber connectors.

TL-4 Railing

The TL-4 railing included a glulam timber rail with curb, as shown in Figure 6. The glulam timber rail with curb consisted of a single glulam timber rail mounted on a sawn lumber post and was a modification of the curb system tested at PL-1. 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.

Low Performance Railing

The low performance railing developed for impact conditions less than TL-1 included a low-height timber curb rail, as shown in Figure 7. Three geometries were considered for the curb rail — a square shape, a rectangular shape, and a trapezoidal shape. The curb rail was constructed with sawn lumber and supported with scupper blocks. The curb and scupper blocks were connected to the bridge deck with bolts.

Concluding Remarks

This program clearly demonstrates that crashworthy railing systems are feasible for longitudinal 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 deck wood bridges has been overcome.

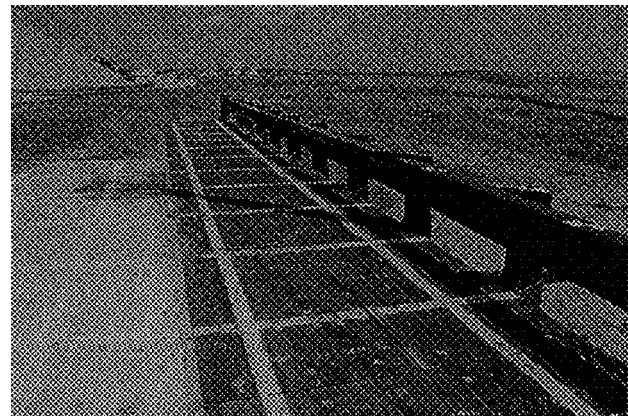
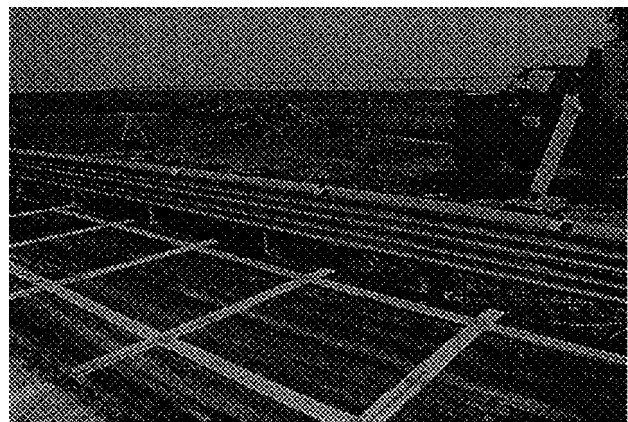
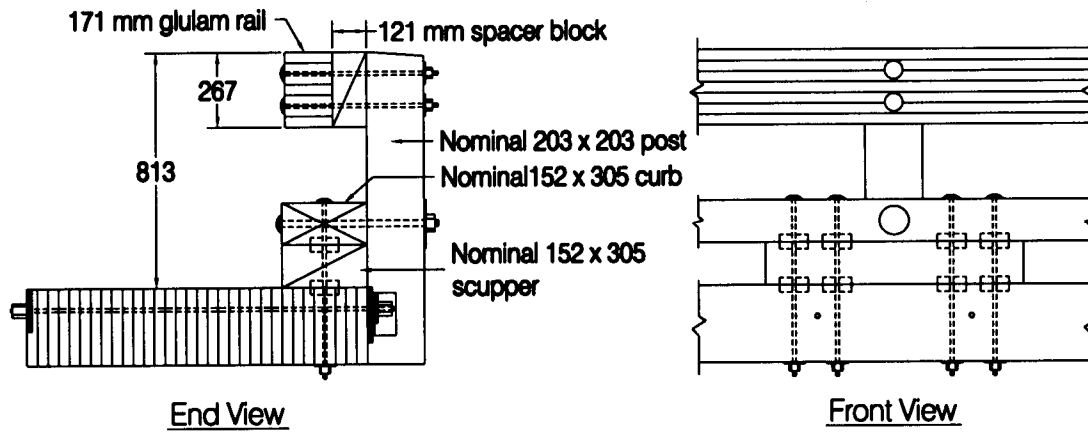
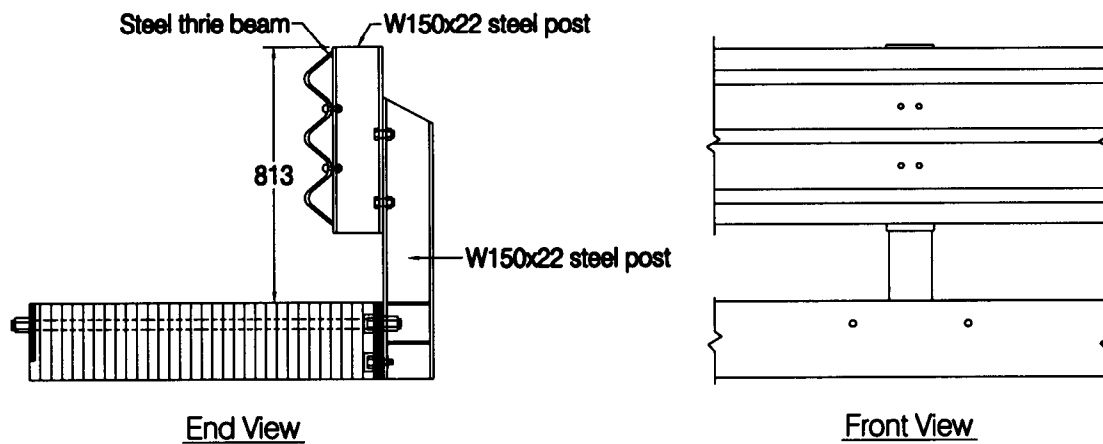


Figure 1 — Photographs of bridge railings successfully crash tested to AASHTO PL-1 (photographs taken prior to testing).

Glulam Timber Rail with Curb



Steel Rail



Glulam Timber Rail without Curb

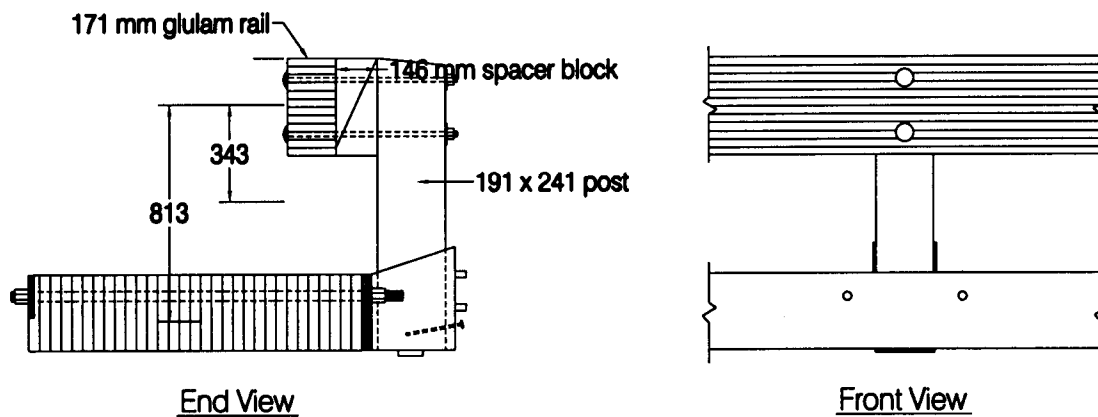


Figure 2 – Drawings of bridge railings successfully crash tested to AASHTO PL-1. Drawing units are millimeters

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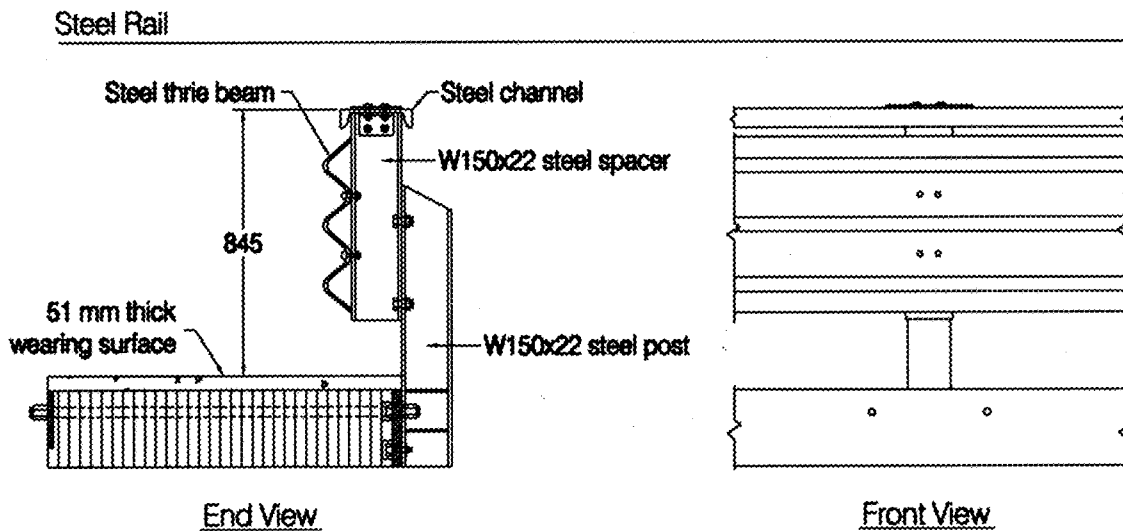


Figure 3 – Steel thrie beam bridge railing successfully crash tested to AASHTO PL-2 (photograph taken prior to testing). Drawing units are millimeters.

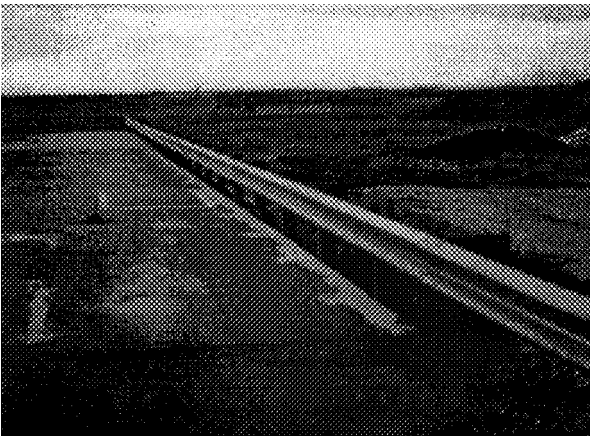


Figure 4 – Photographs of bridge railings successfully crash tested to NCHRP 350 TL-1 (photographs taken prior to testing).

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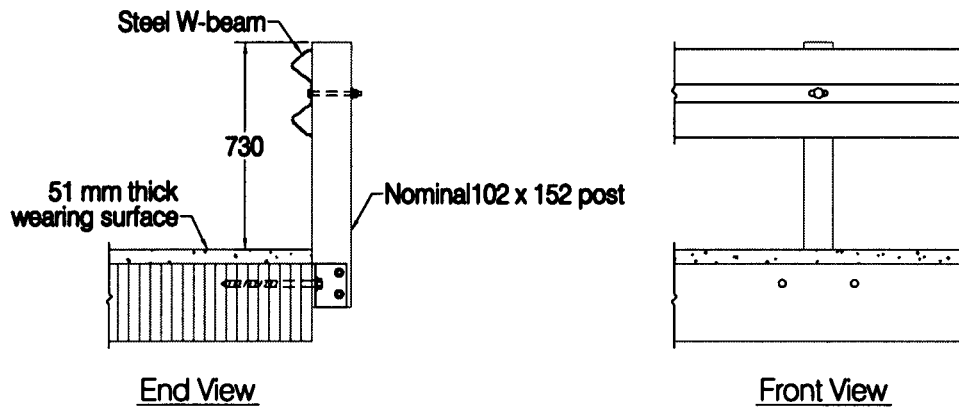
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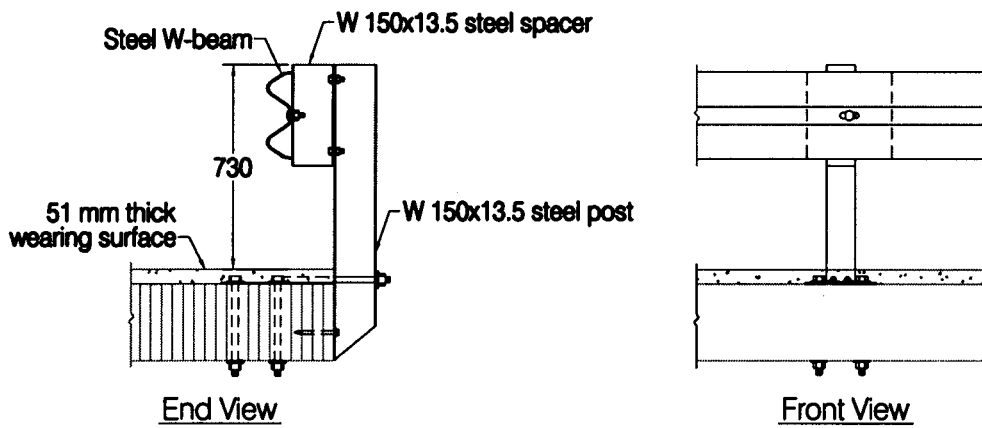
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18. Ritter, M.A., Lee, P.D.H., Faller, R.K., Rosson, B.T., and S.R. Duwadi, *Plans for Crash Tested Bridge Railings for Longitudinal Wood Decks*, General Technical Report No. FPL-GTR-87, United States Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, Wisconsin, September, 1995.

Flexible Steel Rail



Semi-Rigid Steel Rail



Curb-type Rail

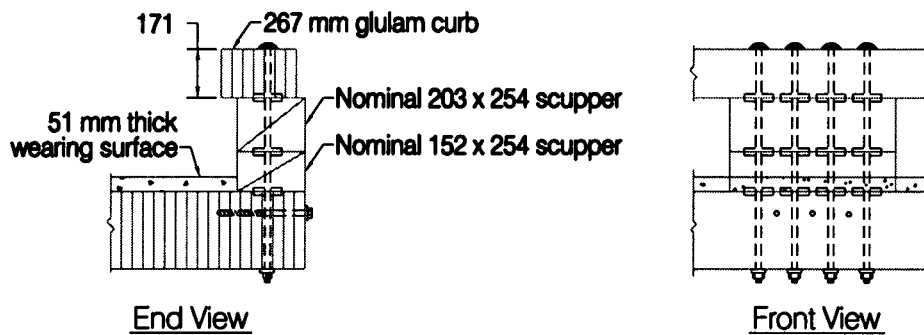
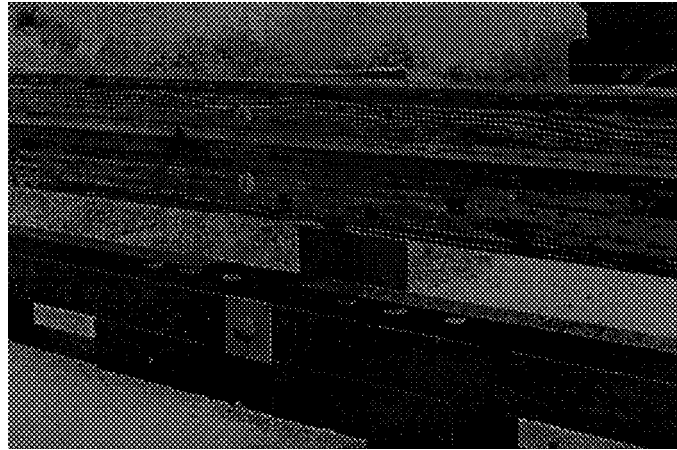


Figure 5 – Drawings of bridge railings successfully crash tested to NCHRP 350 TL-1. Drawing units are millimeters.



Glulam Timber Rail with Curb

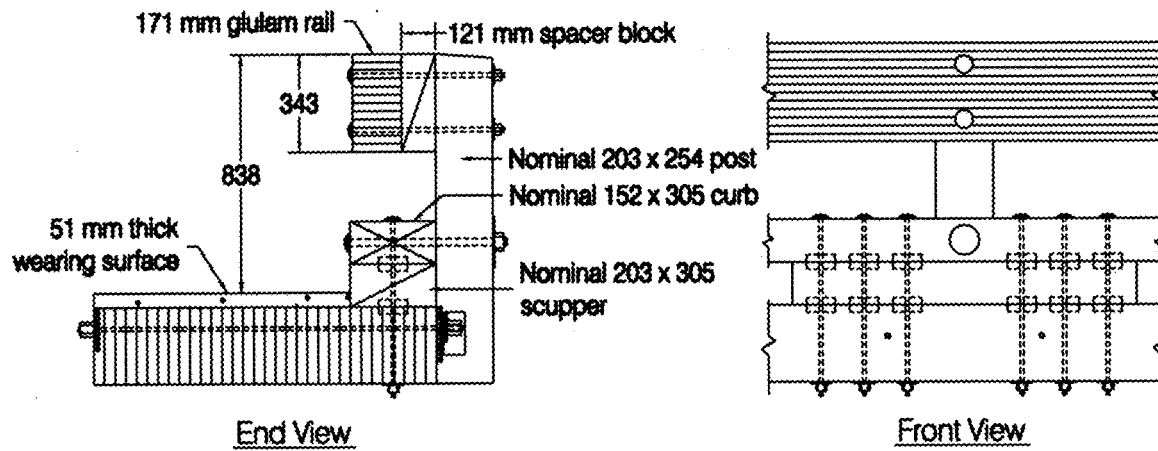
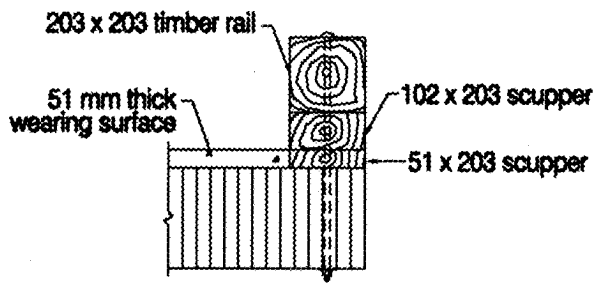
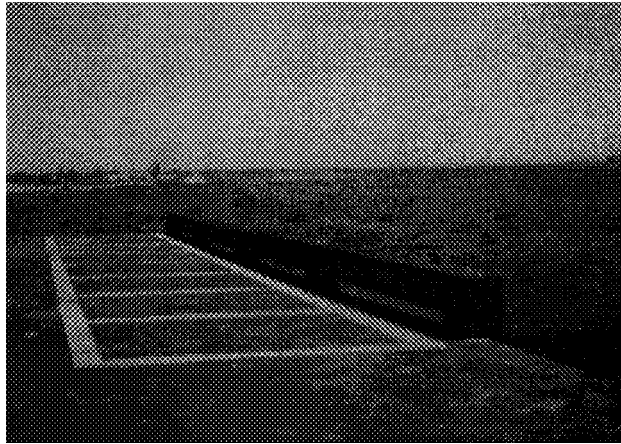
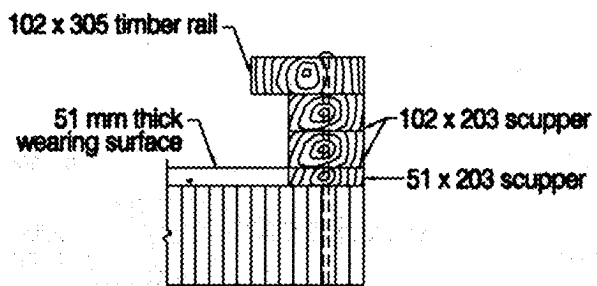


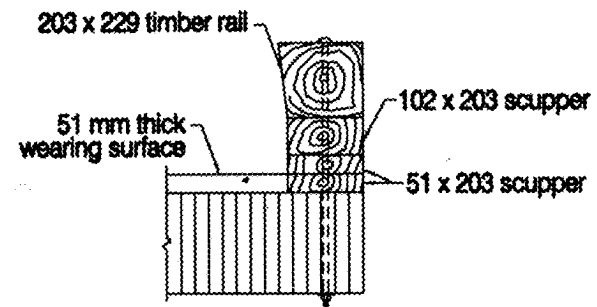
Figure 6 – Glulam timber bridge railing successfully crash tested to NCHRP 350 TL-4 (photograph taken prior to testing). Drawing units are millimeters.



End View



End View



End View

Figure 7 – Low performance bridge railing successfully crash tested with a 3/4-ton pickup truck at 15 mph and 15 degrees (photograph taken prior to testing). Drawing units are millimeters.

In: Ritter, M.A.; Duwadi, S.R.; Lee, P.D.H., ed(s). National conference on wood transportation structures; 1996 October 23-25; Madison, WI. Gen. Tech. Rep. FPL- GTR-94. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.