Composite-Reinforced Timber Highway Guardrail: Development and Structural Testing

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Summary

For aesthetic reasons, various government agencies install timber guardrails on scenic highways and roads in place of conventional steel guardrails. Most acceptable timber guardrails rely on a continuous steel backing member to carry the large tensile forces caused by vehicle impact and transfer load to the posts. However, these guardrails are relatively expensive and heavy due to their use of large, solid-sawn timber sections with steel reinforcing. This study focuses on the development and structural testing of a novel timber guardrail that consists of a hardwood glued-laminated member strengthened with a fiber-reinforced polymer (FRP). The FRP acts in place of steel to carry the impact-induced tension, and the guardrail is significantly shallower and lighter than conventional timber guardrails. Details regarding the analysis of the guardrail response under vehicle impact, the development and testing of a unique and easily installed rail-to-rail field splice connection capable of carrying the tensile forces caused by vehicle impact, and the evaluation of guardrail durability in exterior conditions are given. In addition, the structural behavior of the guardrail was assessed using experiments that produced the simultaneous tension and flexure forces expected during vehicle impact using a single hydraulic actuator and a three-point bending test apparatus. The performance of the guardrail exceeded expected demands, and based on the development and testing documented in this paper, the FRP-reinforced timber guardrail is expected to be capable of passing required vehicle crash tests. Future research should more thoroughly address bond durability, especially when preservative treatments are used.

1. Introduction

Guardrails are designed to protect motorists from roadside hazards. Various designs of steel W-beam guardrails are commonly used in the United States because of their low cost, proven effectiveness, ease of installation, and durability. However, along scenic roads, timber guardrails are often considered a more aesthetically pleasing alternative. An example of an existing timber guardrail in use in the United States is the Merritt Parkway guardrail, which consists of a large solid-sawn piece of softwood with a steel backing plate (Lohrey et al 1997). In most timber guardrails, the steel serves as a continuous tension member and maintains the structural integrity of the rail during a vehicle impact. Unfortunately, timber guardrails are generally expensive due to their use of large solid-sawn timbers. Further, they are relatively heavy and require a large crew or a truck-mounted crane for installation.
The objective of this study was the development of a lightweight, cost-effective timber guardrail that utilizes glued-laminated (glulam) members fabricated from inexpensive, low-grade hardwoods commonly found in the Northeastern U.S. such as red maple and beech. A novel feature of the guardrail is its use of bonded fiber-reinforced polymer (FRP) reinforcing in lieu of steel to carry the tensile forces developed during vehicle impact, which significantly reduces rail weight.

A major requirement for the rail design included a demonstrated high likelihood of passing the NCHRP Test Level 3 (TL-3) crash test (NCHRP 1993), which must be completed prior to highway use of the rail. In order to meet this requirement, it was necessary to analyze the guardrail response under vehicle impact, develop a practical rail-to-rail field splice connection capable of transferring the large tensile forces caused by vehicle impact, and evaluate the guardrail’s durability for exterior use. In addition, the structural performance of the guardrail was assessed using unique structural tests designed to produce the combined tension and flexure forces produced in a guardrail by vehicle impact.

2. Guardrail Details and Estimation of Guardrail Response

2.1. Guardrail Cross-Section

The proposed guardrail designed and tested in this study is a reinforced glued-laminated (glulam) rail section fabricated in a brickwork layup from readily available mixed hardwoods (primarily red maple) as shown in Figure 1. The guardrail is reinforced with a 3.5mm thick, unidirectional E-glass FRP, which serves both as a tension ribbon and as flexural tension reinforcement during vehicle impact. Preliminary flexural tests of reinforced glulam beam specimens showed that this volume of FRP tensile reinforcing gives the rail substantial bending ductility (Botting 2003). The height of the rail section was fixed at 254mm, which is an intermediate value for timber guardrails currently in use in the United States.

The glulam sections were fabricated in-house using high clamping pressures and a phenol-resorcinol formaldehyde resin commonly used in glulam beam manufacture. The FRP was a commercially available, E-glass epoxy bar stock produced by Gordon Composites, Inc. that was bonded to the glulam with epoxy resin. This particular combination of FRP and epoxy has been used successfully in the past to reinforce softwood glulam beams (Weaver et al. 2004; Dagher and Lindyberg 2003) and the bond durability was assessed as detailed later in this paper. See Botting (2003) for a complete description of the guardrail manufacturing procedures.

2.2. Estimation of Guardrail Design Forces

The first phase of this study involved simulating vehicle impact on the rail system to estimate the tension developed during impact (the critical design load) as well as the lateral deformation of the rail and vehicle decelerations. The response of the guardrail system was evaluated using Barrier VII, a 2D dynamic finite element program originally developed by Powell (1973) that has been widely used to evaluate guardrail system performance. The NCHRP Report 350 Test Level 3-11 (TL-3) crash test (NCHRP 1993), which requires rail impact with a 2000kg pickup truck was simulated with Barrier VII. This test was chosen from all TL-3 tests because it was expected to be critical for the structural integrity of the rail section. Further details on the simulations can be found in Botting (2003) and Davids et al. (2006).
Rail cross-sections with various depths \( d \) (see Figure 1) ranging from 76mm to 156mm were considered to study the effect of specimen geometry on response. We note that the cross-section is similar to the crash-tested Merritt Parkway Guardrail when \( d = 156 \text{mm} \). In addition, an impact simulation was also performed for a steel W-beam section to establish that displacements and decelerations were acceptable relative to this proven alternative. As expected, the thinnest rail \( (d = 76 \text{mm}) \) responded most similarly to the W-beam rail, which has a very small flexural stiffness. For all cross-sections, large axial forces are induced in the rail, and the thinnest rail must sustain an axial force of 242kN. This is a critical design value, since it must be carried by the field splices between sections of rail.

Based on the simulation results, the need to minimize guardrail weight, and the better ductility expected with larger ratios of FRP volume to wood volume, the 76mm-deep rail was selected for experimental characterization. We note that a 3.66m-long x 76mm-deep section of rail will weigh approximately 300N, and could be easily installed by three workers without the aid of a crane.

3. Field Splice Connection Development and Testing

3.1. Overview

This section details the strength and durability testing of the splice connection, which is critical to the performance of the guardrail system. The 242kN tensile force developed during impact must be transferred by the FRP through the guardrail system and into the posts and soil. Assuming a standard post spacing of 1.83m, practical construction and maintenance requirements dictate that one rail section span two post spaces. Thus, a field splice is required between rail sections that must be capable of transferring 242kN of tension. This presented a unique challenge, since the ideal connection requires only simple field bolting, yet it is very difficult to develop forces of this magnitude in bolted timber and FRP connections. We note that the fiber orientation of the FRP laminate could be varied in the region of the connection to increase the bolted connection capacity. It may also be possible to increase the connection strength by adding additional FRP layers with appropriate fiber architecture between wood laminations or on the exterior of the rail in the region of the connection. However, such approaches are labor-intensive, costly, and prior research (Soltis et al. 1998; Chen 1998) does not indicate that they can produce the required connection strengths.

Our solution was to develop the connection shown in Figure 2, which relies on a 13mm-thick steel plate that can be bonded to the FRP in a manufacturing facility or maintenance shop with a thick layer of epoxy under controlled conditions. The field-installed connection consists of a simple steel-to-steel splice with six 19mm-diameter bolts. A study by Boone (2002) showed that it is possible to transfer large forces between FRP and steel through shear in a thick epoxy bond. Based on this work, we chose SIA Adhesives E2119 epoxy (manufactured by Sovereign Specialty Chemicals in the United States) to bond the steel plate to the FRP. This adhesive is cost-effective, ductile, and has high shear strength at a reasonable bond line thickness.

3.2. Field Splice Strength Testing

A total of six specimens were tested in direct tension using a 1780kN capacity Baldwin-Satec testing frame to determine the splice capacity as shown in Figure 3. Load and crosshead displacement were recorded, and foil strain gages were applied to the FRP to monitor peak tensile strains. A potentially critical parameter governing connection performance is bolt torque, since bolt
pre-tension compresses the steel-FRP epoxy bond line and can thus be expected to increase its load capacity by reducing tensile peeling stresses at the epoxy bond line. Further, significant losses in bolt tension can be expected within a few days or weeks after installation due to compressive creep of the glulam.

To assess this effect, three different initial bolt torques were used in the testing program, with two nominally identical specimens tested at each torque level. Specimens T1-T2 were constructed with a bolt torque of 136kN-m, which is the torque that initiated crushing of the wood beneath the nut and washer. Specimens T3-T4 were torqued to 54kN-m, which is 60% less than 136kN-m, and specimens T5-T6 were torqued to 27kN-m, an 80% reduction from 136kN-m. A load rate of 22kN/minute was maintained until the specimen load reached about 350kN, and the remainder of each test was run in displacement control from that point forward for safety reasons.

Failure for each specimen was characterized by slip of the steel-FRP epoxy bond line accompanied by the bolts nearest the end of the specimen tearing out the end of the FRP-reinforced glulam. The effects of bolt torque on capacity are clear from the results: the average capacity of specimens T1-T2 was 462kN, the average capacity of T3-T4 was 427kN, and the average capacity of T5-T6 was 383kN. However, all specimens carried at least 53% more load than the estimated peak impact-induced tension of 242kN. The initial slip in the connection was generally smaller with the largest bolt torque, which is attributed to increased friction due to higher clamping forces. It is worth noting that the ultimate design capacity of the six 19mm-diameter A490 bolts used in the connection is 531kN (AISC 2002), only 15% greater than the maximum observed strength. Peak strains measured in the FRP ranged between 5100 µstrain (specimen T6) and 6500 µstrain (specimen T2); these values are well below the expected strain at failure of the FRP. Given the excellent performance of the connection, no modifications were made to the original design.

3.3. Field Splice Durability Testing

Testing was performed on the splice connection to study the durability and delamination potential of the splice connection and guardrail due to moisture changes caused by exterior exposure. This is particularly critical given the use of the bonded steel splice plate, since while the wood shrinks and swells significantly under moisture changes, the steel undergoes no dimensional changes, causing significant stresses at the steel-FRP bond line.

Given the unique nature of this connection and guardrail, there is no universally accepted standard test for evaluating the effect of moisture movement. The ASTM D1101 Test Method A delamination test (ASTM 2002) was chosen to evaluate the effect of moisture changes, since it applies for glulam beams intended for exterior use and subjects the specimen to severe moisture cycling through sequential pressurized wetting and drying cycles.

In accordance with ASTM, tests were performed on 76mm-long rail cross-sections. Three of the six delamination specimens included the bonded steel splice plate and three control specimens were tested without the steel splice plate. Despite the very large swelling of the maple glulam as seen in Figure 4, none of the specimens with the bonded steel splice plate exhibited measurable delamination at either the steel-FRP bond or the wood-FRP bond, and therefore passed according the criteria of ASTM D1101. The good performance is likely due to the thick steel-FRP epoxy bond
and the FRP itself acting as a compliant layer between the steel and the wood, which helped dissipate the high shear stresses that would otherwise develop if the steel was bonded directly to the wood with a thin layer of adhesive. However, we note that further testing should be performed prior to field installation, and that the use of preservative treatments may affect durability.

4. **Bending-Tension Tests**

While the tension test results indicate that the field splice connection has sufficient capacity, it is critical that a section of the guardrail be tested under the simultaneous tension and bending caused by vehicle impact. Based on the results of the crash simulations discussed earlier, a critical question is whether or not the rail will be capable of acting as a tension ribbon once its flexural capacity is exceeded.

4.1. **Development of Test Method and Design of Test Apparatus**

Our solution for achieving the simultaneous application of tension-producing axial and transverse bending forces was to design a self-reacting steel truss that produced tension in the guardrail due to the shortening caused by bending of the guardrail under transverse load. This required the use of only one actuator with a simple test frame, where a three-member truss as shown in Figure 5 supports each end of the guardrail.

The splice connection detailed previously was used to attach each end of the guardrail specimen to the truss supports. The plane of the splice connection was horizontal, and the downward transverse load $F'$ is bending the guardrail about its weak axis with the FRP in tension. To ensure that tension is developed quickly under transverse loads, the test frame is self-reacting in the horizontal direction and shimmed tightly prior to the start of each test, and all truss connections are made with true pins installed with slight interference fits. Since the truss members remained linearly elastic during the test, the tension induced in the guardrail could be accurately calculated from strains measured in each diagonal, the elastic modulus of steel, and the cross-sectional area of the diagonals.

The challenge in designing this test apparatus was sizing the truss diagonals to produce the desired level of tension at a target transverse force. To achieve this, a geometrically nonlinear structural model was used to simulate the response of the guardrail and account for the $2^{nd}$-order effect of curvature-induced rail shortening. All members were assigned nominal material properties, and assumed to behave linearly elastically. This assumption is valid until initial tensile fracture of the
glulam portion of the guardrail, which generally occurred at about half the peak applied load as discussed in Section 4.3.

4.2. Instrumentation and Test Protocol

A total of three nominally identical specimens (denoted BT1 – BT3) were tested. To monitor longitudinal tensile strain in the FRP reinforcing, four foil strain gages were bonded to the FRP of each specimen. One pair of gages was located at the center of the guardrail specimen span, and one pair at a quarter-point of the clear span between splice connections. Each gage was located 63mm from the edge of the FRP plate. To monitor tension in the truss diagonals, and thus get an accurate estimate of the guardrail tension, each diagonal member was instrumented two strain gages, one on each side at the center of the member.

Loads were applied using a 500kN actuator mounted below the laboratory strong floor, and the actuator displacement and applied load were recorded during the tests. A neoprene pad was sandwiched between the radiused hardwood load head and the specimen to provide more uniform load distribution. The load-deformation characteristics of the rubber pad were estimated from separate compression tests, and subtracted from the actuator displacement to give a reasonably accurate mid-span displacement of the guardrail specimen.

Immediately prior to starting the transverse load application, each splice connection bolt was torqued to 136kN-m to minimize connection slip. Additionally, shims were driven into the small gaps between the horizontal compression members on the reaction floor to ensure that the test apparatus would self-react over the full range of applied loads.

4.3. Bending-Tension Test Results

The behavior of all three specimens was very consistent. Figure 6 shows experimentally observed response for specimen BT2. The stiffening response due to the tensile force produced by the truss supports is evident in the load-displacement response. The model predictions from the 2nd-order structural analysis used to design the supporting trusses are also shown for comparison at lower loads. The model predicts the observed stiffening response, and agrees well with the test results. Specimens BT1, BT2 and BT3 were able to sustain peak transverse loads of 179 kN, 202kN and 179kN, respectively. In all three tests, the rail sections were still carrying load when the test was halted, since larger loads would likely have overloaded the testing frame. Indeed, the weldment to which the specimens were spliced was bent while testing specimen BT2. Initial tensile fracture of the glulam occurred at an applied load of 83kN in specimen BT2, which is evident in Figure 6. Initial tensile fractures occurred at similar applied load levels in specimens BT1 and BT3.

Based on the measured strains in the truss diagonals, specimen BT2 was subjected to a peak tensile force of 461kN. This is 90% greater than the estimated impact-induced tension of 242kN, and nearly equals the maximum average splice connection strength measured during the direct tension tests. The peak induced tension forces in specimens BT1 and BT3 were 435kN and 422kN, respectively, slightly less than the 461kN observed for specimen BT2. The model accurately predicted the relationship between applied transverse load and axial tension induced by the truss supports. The peak strain recorded in the FRP ranged between 62% and 76% of the expected FRP strain at failure for the three tests.

Overall, the three specimens exhibited very consistent response, and sustained transverse loads and induced axial forces well in excess of those expected during vehicle impact. It is also worth noting that the Barrier VII models detailed earlier predict a maximum transverse deflection of the rail relative to adjacent posts of 140mm, and all of the bending-tension specimens sustained peak deflections at least 40% greater than this value. This indicates that the rail section has sufficient ductility.
5. Summary and Conclusions

A guardrail has been developed that consists of a 254mm-high by 76mm-deep hardwood glulam reinforced on one face with 3.5mm of unidirectional E-glass FRP. The structural demands on the guardrail were estimated using the computer program Barrier VII. Durability tests were conducted that demonstrated that the guardrail will potentially perform well in an outdoor environment if properly preservative treated. A unique bonded tension splice was developed that allows steel-FRP bonding to be performed under controlled shop conditions and simple field-bolted assembly. The splice was tested for strength and delamination resistance, and found to have good performance. Additionally, an innovative test was developed to simultaneously apply expected levels of flexure and tension in a guardrail specimen using a single hydraulic actuator and three-point bending. The measured capacities of the guardrail under combined tension and flexure exceeded the expected demands in all test specimens.

Based on the comprehensive evaluation and testing conducted in this study, we conclude that the FRP-reinforced timber guardrail as designed has a high probability of passing a TL-3 crash test and gaining acceptance for use in highway applications under the requirements of NCHRP Report 350 (NCHRP 2003). However, there are several aspects of guardrail fabrication and performance that warrant further study. First, the adequacy and long-term durability of the FRP-wood bond should be assessed with more rigorous testing, and qualified for specific preservative treatment systems. Second, based on the observed tensile capacities of the splice connection, it may be possible to reduce its size and the bolt capacities prior to crash testing the rail. However, further laboratory testing should first be conducted to verify that a re-designed connection is adequate. Finally, the fabrication cost of the rail may be reduced if the brickwork layup shown in Figure 1 is abandoned in favor of horizontally stacked laminations with a constant 76mm width. However, the FRP-wood bond quality may be detrimentally affected by the use of horizontally stacked laminations, and this must be investigated further.

![Fig 6 Response of Bending-Tension Specimen BT2](image)
6. Acknowledgements

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7. References


