Dynamic Impact Load Tests of a Bridge Guardrail System

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Acknowledgements

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Disclaimer

ABSTRACT

Bridge guardrail systems are vital components of roadway bridges. Timber bridge guardrail systems existing at the time of the study described herein met federal bridge design code requirements for static loads. However, at the time of the study described herein, they had not been subjected to either laboratory impact test loads or federal crash tests. A structural concept as an alternative to conventional bridge guardrail system for application in timber bridges was conceived in prior related research on an innovative timber bridge system. Initial laboratory static load tests were conducted on a prototype bridge guardrail specimen, and outcomes were encouraging. However, some improvements were needed in the test set-up. This paper presents the results of the subsequent study, which was conducted after making the improvements. The results showed the main component of a steel hanger performed without significant distress. Applied transverse ramp load was favorably distributed into the transverse floor beam segment of the specimen, and no noticeable damage resulted at q load level comparable to the ultimate load implied in bridge design code requirements of the American Association State Highway and Transportation Officials. An exploratory test under pendulum impact load testing showed the hanger and its connection system performed with no major damage. The main bridge girder sustained structural damage, but extreme handling methods needed to move the specimen into the load position and a reduction in its depth prior to testing may have damaged the girder and contributed to that outcome being suspect. It is recommended that the system be tested under crash loading in the future, but built where no major handling is needed before the crash test itself.

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EXECUTIVE SUMMARY

This report describes the conduct and outcomes of laboratory studies of an innovative timber bridge guardrail system conceived as an alternative to conventional systems used in timber bridges. Considerable attention has been placed on the deteriorated state of bridge highway infrastructure in the United States. America's bridges are aged and aging, and many are structurally deficient and/or functionally obsolete. The vast majority (over 140,000) of inadequate bridges is on non-national highway system roadways and many involve timber construction. Thus, many rural timber bridges are in inadequate condition. Substantial investments are needed to improve the state of these bridges amid limited local funds. This situation motivated the Federal Highway Administration and U.S. Forest Service to foster extensive research and development on innovative concepts for timber bridges for secondary roads. Among the priority needs was addressing a lack of adequate timber bridge guardrail systems.

Analytical design requirements for guardrails were contained in past American Association of State Highway and Transportation Officials (AASHTO) bridge design codes ("Standard" 1989 and later until the most recent code). About a decade ago, the AASHTO developed criteria for the conduct of (then optional) full-scale crash testing for acceptance of bridge guardrail systems. Physical performance tests became mandated in the current AASHTO bridge design code. Loads to be used in analytical development of new bridge guardrails system are also provided for configuring systems so as to have the potential of surviving full-scale crash tests.

The National Cooperative Highway Research Program, NCHRP Report 350, presents specific impact conditions for vehicle mass, speed, approach angle, and location on the safety feature to be hit. Static and dynamic (gravitational pendulum and bogie vehicle) tests are typically used before a decision is made to embark on an expensive full-scale crash test. The research described in this report was part of the development of a novel "anisotropic-grid timber bridge" (which includes the innovative bridge guardrail) for short to medium spans. Earlier comparative ramp load tests of a prototype of the innovative bridge guardrail system and a counterpart conventional system were encouraging, suggesting a high superiority of the former. However, improvements in the test set-up were needed as well as exploratory dynamic load tests using a pendulum. This report describes the tests conducted after making the improvements.

Ramp load applied to a specimen showed no visible evidence of failure. The applied lateral load at a post was distributed mostly into the transverse floor beams directly below the post and the two adjacent to it. However, the specimen was only a portion of the bridge and had simulated conditions of end movement. The behavior suggested a more uniform distribution would occur in an actual bridge. A low impact pendulum provided approximate insight as to the impact time of a pendulum impact. Higher impact tests using a massive concrete block showed the hanger/transverse beam connection behaved with no meaningful distress. However, the main girder sustained some cracks along the length. It had been considerably changed in depth (based on the successful ramp test) and handled in an extreme manner to put it into position for the pendulum tests. These factors may have contributed to unseen fractures that could have compromised the girder. While mixed results occurred, the overall sense is that this type of specimen should be subjected to vehicle crash testing for further understanding of its behavior.

1. INTRODUCTION

Considerable attention has been placed on the deteriorated state of bridge highway infrastructure in the United States. America's bridges are aged and aging and some have failed dramatically in recent decades. Statistics cited by the authors in other publications (Gutkowski et al. 1994, Gutkowski, et al. 2003) indicate the severity of the situation. Data in the National Bridge Inventory (U.S. Department of Transportation 2000) indicated that about 165,000 of the nation's 590,000-plus highway bridges are categorized as either structurally deficient or functionally obsolete. The vast majority (over 140,000) of these inadequate bridges are on non-national highway system roadways. Substantial investments are needed to improve the state of these bridges. About $6 billion (in 1997 dollars) is needed annually just to maintain the backlog (U.S. Department of Transportation 1999). To eliminate that backlog and address all accruing needs until 2017 requires about $11 billion annually.

1.1 Rural Bridge Dilemma

Local highway officials face a dilemma with respect to inadequate funding for bridge improvements and replacements (U. S. Department of Agriculture 1989). The Highway Bridge Repair and Replacement and Rehabilitation Program (HBRRP) provides annual federal funds for bridge construction needs. The portion of HBRRP funds allocated to secondary roads is limited, and such use is at the discretion of each state. Thus, many rural bridges remain in inadequate condition. In particular, over 48% of county bridges were reported as having bridges with nominal guardrail systems, either just meeting minimum tolerable standards or worse.

The above situation motivated extensive research and development on innovative concepts for timber highway bridges (Gutkowski 1997, 1999, Duwadi et al. 2001). The use of timber in bridge applications has the potential to significantly increase the construction of new timber bridges on rural roads. Implementation of some of the new timber bridge technologies has already been occurring on secondary roads. The Federal Highway Administration and U.S. Forest Service have been involved in an initiative to foster increased use of timber for highway bridges and other transportation structures (Ritter 1992, 1997, 2001).

Wipf et al. (1993) enumerated numerous research and development needs for improving timber bridge technology and engineering methodology. Among them were needs related to improving timber bridge guardrail systems. Lack of adequate timber bridge guardrail systems has been one deterrent to greater implementation of timber bridges for rural bridge needs. This paper presents the outcomes of laboratory studies of a timber bridge guardrail system, which constitutes an alternative to conventional systems used in timber bridges.

2. EVALUATION OF BRIDGE GUARDRAILS

The American Association of State Highway and Transportation Officials (AASHTO) has given particular attention to improving bridge appurtenances. The use of analytical methods for configuring bridge guardrails for structural resistance had been the design code approach. Analytical requirements are contained in past AASHTO bridge design codes ("Standard" 1989 and later until the present code). About a decade ago, AASHTO took a significant step to improve the future safety of bridge guardrails. Specifically, criteria for the conduct of full-scale crash testing for acceptance of bridge guardrail systems were developed. As cited subsequently, published guidelines and standards now exist for the conduct of full-scale crash testing for acceptance of guardrail systems. These documents include other acceptable load testing methods (e.g., impact tests based on pendulum devices) as well. The test procedures are an alternative to the use of analytical methods for configuring guardrails for structural resistance. However, physical performance tests became mandated in the current AASHTO bridge design code. Loads to be used in analytical development of new bridge guardrails system are also provided, but only as a basis of configuring systems so as to have the potential of surviving full-scale crash tests.

2.1 Performance Test Requirements

Safety performance of bridge guardrails can be judged on the basis of three factors: structural adequacy, occupant risk, and vehicle trajectory after collision (Ross et al. 1993). Structural adequacy is generally the primary factor to be evaluated. A guardrail satisfies this evaluation criterion if it redirects the impacting vehicle without further incident or permits its controlled stop in a predictable manner. Occupant risk relates to the degree of hazard to which occupants in the impacting vehicle are subjected. It is assessed by the response of a hypothetical unrestrained front seat occupant whose motion relative to the occupant compartment is dependent on vehicular accelerations. The occupant is assumed to move through space until striking a hypothetical instrument panel, windshield, or side structure, and subsequently is assumed to experience the remainder of the vehicular acceleration pulse by remaining in contact with the interior surface. Vehicular trajectory hazard is a measure of the potential of post-impact trajectory of the vehicle to cause a subsequent multi-vehicle accident. This could either subject the other vehicles to undue hazard or subject the occupants of the impacting vehicle to secondary collisions with other fixed objects. To minimize the hazard, the vehicle trajectory and final stopping position should intrude no more than a specified maximum distance, if at all, into adjacent or opposing traffic lanes.

2.2 Crash Testing

Full-scale crash testing has been and will continue to be the most effective method of realistically evaluating the safety performance of guardrails, median barriers, bridge railings, crash cushions, break away supports, truck-mounted attenuators, work zone traffic control devices, and other hardware. A number of published documents describe the evolution of recommended procedures for guardrail crash testing. Procedures for full-scale vehicle crash testing of guardrails were first published in *Highway Research Correlation Services Circular 482* in 1962 (NCHRP 1962). This circular specified vehicle mass, impact speed, and approach angle for crash testing. *Transportation Research Circular 191* published in 198l addressed a number of minor changes requiring modified treatment of particular problem areas. Later, *Circular 191* was expanded and published as *NCHRP Report 230*, "Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances" (Michie 1981). An updated document, *NCHRP Report 350,* was published in 1993 (Ross et al. 1993). *NCHRP Report 350* presents specific impact conditions for vehicle mass, speed, approach angle, and location on the safety feature to be hit. Standard test vehicle types are defined for mini-compact and sub-compact passenger cars, standard 3/4-ton pickup trucks, pickup trucks, single-unit trucks, and tractor-trailer cargo trucks. Approach angles vary from 0 to 25 degrees, and impact speeds range from 20 to 60 mph. Pursuant to the need for crashworthy timber bridge guardrails, an extensive experimental program was undertaken by various researchers (Faller et al. [1993, 1995, 1996, 1999, 2000], Rosson et al. [1995], Raju et al. [1994], and Fowler [1997]). The results of these tests contributed to the development of standard plans for several successfully crash tested conventional timber bridge guardrail systems (Ritter et al. 1998).

2.3 Experimental Tools

Two other available types of tests to experimentally evaluate and study a bridge guardrail system are permitted: static tests and dynamic tests. These tests are typically used during the early stage of bridge guardrail development when certain critical details and connections require strength/deflection evaluation. They provide insight about the guardrail system before a decision is made to embark on an expensive full-scale crash test.

Static testing is typically used to compare competing design rails. Static tests have one of the following objectives: 1) demonstrate guardrail performance under simulated environmental loading, 2) evaluate ultimate strength of critical connections, or 3) develop load/displacement properties for subsequent computer model simulations.

Dynamic test methods include the use of either a gravitational pendulum, drop mass, scale model, or bogie vehicle. A gravitational pendulum involves an impact test using a striking mass that swings in a circular arc suspended by cables or by rigid arms from a main frame. The specimen is generally mounted in an upright manner. Gravitational pendulums typically evaluate performance at impact speeds of 25 mph or less. Drop mass facilities utilize a rigid striking mass that falls vertically to the impact point. The specimen is mounted horizontally to a rigid test fixture plate. Typically, impact speeds are low and specimen sizes are very limited. Scale model testing involves constructing models of both the highway safety feature under investigation and the test vehicles to a reduced scale. The complexities of modeling automobile metal crushing, tire-pavement interaction, and suspension behavior have limited the application of scale modeling procedures. A bogie vehicle is a movable structure mounted on four wheels and the mass equivalent of a selected passenger vehicle. The bogie vehicle is steered by rails, guide cables, remote control, or other means to strike the specimen at the selected impact angle and velocity.

2.4 Basis for the Research Study

Prior to the HBRRP, extremely limited funds were available for rural bridge construction. This contributed to extensive neglect of needs. One evident negative outcome on rural bridges, especially older ones, was the prevalent use of nominal bridge guardrails (U.S. Department of Agriculture 1989). The description “nominal” is used in a structural sense, i.e., it implies many bridge guardrails below current minimum standards are presently in place. Indeed, some have been involved in vehicular accidents where loss of life occurred. In some states, local jurisdictions purposely omit bridge guardrails on lightly traveled rural roads to avoid costs (U.S. Department of Agriculture 1989). Thus, upgrading of bridge guardrails is especially needed on rural roads.

The need for a bridge guardrail pre-testing procedure exists because federal-level crash testing of guardrails has certain drawbacks, the principal one being expense. One must configure full-scale bridge guardrails or bridges and use actual vehicles (autos, trucks, buses, etc.) to conduct performance tests. Different levels of certification exist, too. The tests constitute a “go - no go” philosophy that tends to limit structural engineers to the use of “standard configurations” in practice. For development of new guardrails, crash testing can be an imposing constraint, since extensive development costs can be incurred repeatedly if a series of improved prototype systems continue to fail a federal crash test. It is advantageous to have less expensive methods to pretest potential new bridge guardrails, e.g., tests done in a laboratory setting before proceeding to a sequence of federal crash tests. Accompanying analytical models are useful in understanding the systems behavior, too. They permit the engineer the flexibility to employ design ingenuity to configure innovative prototype bridge guardrail systems.

3. BACKGROUND

A few timber bridge guardrail systems that existed at the time of the study described herein met federal bridge design code requirements for static loads (AASHTO 1999 and earlier). However, at the time of the study described herein, they had not been subjected to either laboratory impact test loads or federal crash tests. The research described in this report was part of an extensive laboratory study conducted at Colorado State University (CSU), which had the development of a “semi-empirical” mathematical model for bridge guardrail systems as its goal. The intent of that model was to provide an analytical tool and laboratory procedure for use in pre-testing prototype bridge guardrail systems before electing to perform crash tests. A series of studies by Malone et al. (1987), Kalin et al. (1989), and Gutkowski et al. (1990, 1993, 1994) led to a promising result. The main effort was to evolve as simple a general computer model of bridge guardrails as possible that would provide sufficient accuracy of results to be dependable for comparing load-displacement behavior of different guardrail systems. The desired result was verification of the analytical predictions of static load resistance by the conduct of full-scale laboratory tests of a short section of a bridge guardrail-superstructure system.

A sequence of steps was taken to use increasingly more rigorous or detailed mathematical models to achieve the desired result. The intent was to retain the simplest model that proved accurate and minimized the size of the needed laboratory specimen. Initially, a simple continuous beam on elastic supports representation was attempted, but produced unacceptable results. Then a matrix stiffness analysis method was developed which employed empirically measured flexibility coefficients for each of the posts to incorporate their stiffness (Malone et al. 1987). Improvement in the prediction of behavior resulted in better accuracy, but was still deemed to be inadequate. The principal factor was the use of the uncoupled flexibility coefficients for the posts. To improve the model, the posts were modeled in a coupled manner. This outcome proved to be more accurate Kalin et al. (1989), Gutkowski et al. (1990, 1993).

Subsequent research was conducted on the comparative of performance of timber bridge guardrail specimens in a posttensioned and non-posttensioned longitudinal deck bridge (Gutkowski et al. 1994). That paper focused on transverse, longitudinal, and angular load tests of a conventional timber bridge guardrail system, used in that type of bridge. Thus, laboratory studies were conducted on a prototype bridge guardrail system which is an alternative to conventional timber bridge guardrail systems (Favre et al. 1996). Outcomes were encouraging. However, some improvements were needed in the test set-up. This paper presents the results of subsequent study, which was conducted after making the improvements.

3.1 Conventional Bridge Guardrail Systems

A conventional guardrail system for a bridge is depicted in Figure 1. Primary components are an approach guardrail, a bridge guardrail, and a series of posts to which the rails are attached. An anchor system is used to tie the systems into the subgrade at the terminal points of the approach guardrail. The posts of the bridge guardrail are connected directly into the superstructure of the bridge. Different types of standard members are used for the bridge guardrail as well as the connection details to fasten it to the posts. The design engineer selects the configuration based upon the type of bridge superstructure involved and the desired construction materials.

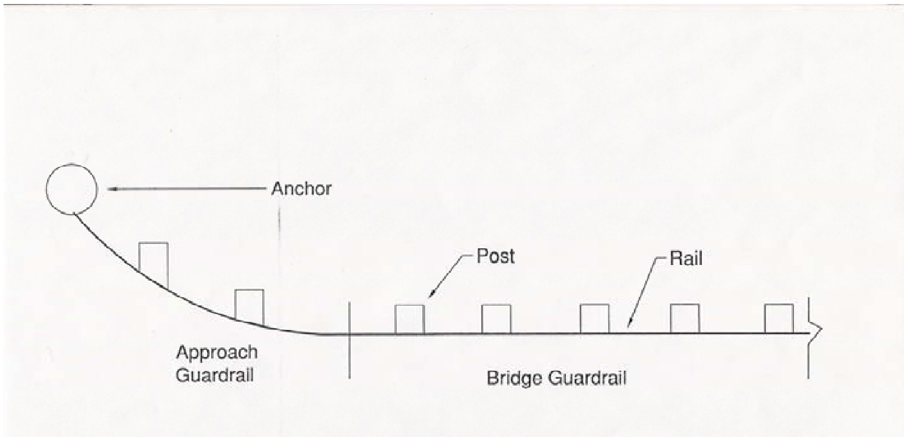

Figure 1 Conventional bridge guardrail system

3.2 Alternative Bridge Guardrail System

The study described herein was also part of the development of an experimental "anisotropic-grid" timber bridge system (Favre et al. 1994, 1996, Natterer 2000) for short to medium span needs. The innovative timber bridge guardrail system (Favre et al. 1996) was included in the concept. In the anisotropic-grid timber bridge system, a through-girder primary system is used. Steel hangers are used to suspend transverse floor beams from the through-girders (Gutkowski et al. 2001). The hangers also hold timber guardrail falsework posts and the steel bridge guardrail in place. Transverse loads applied to the rail are transmitted to the floor beams by bearing of the posts against the steel hangers. The horizontal forces in the floor beams are then transmitted to the abutments primarily by shear in the longitudinal deck placed atop the floor beams.

CSU researchers cooperated with the La Plata County Road and Bridge Department (LPC-RBD) to replace a badly deteriorated steel stringer bridge with an anisotripic-grid timber bridge. The original bridge was located on County Road 243 in Durango, Colorado. It also had experienced impacts to its guardrail terminal posts by logging trucks. Then existing federal design requirements (AASHTO 1989) were used in the design process for the replacement structure. The light weight replacement bridge superstructure permitted the existing damaged concrete abutments to be repaired and left in place. Prior county plans had been to replace them entirely. A Colorado engineering firm made an independent check of the design computations for the replacement bridge on behalf of La Plata County. The field construction and the research involved in its development are described in subsequent technical papers. A reduced size prototype of the complete bridge concept was studied analytically and used in laboratory testing (Favre et al. 1996, Gutkowski et al. 1994, Shigidi 1999). The semi-empirical mathematical model (Malone et al. 1987, Kalin et al. 1989) was used in the study of this prototype bridge guardrail system as well. After its construction, field load testing of the actual County Road 243 bridge was conducted (Favre et al. 1994, 1996, Gutkowski et al. 2001, 2006) demonstrating the acceptable performance of the bridge.

3.3 Scope of the Study

As noted earlier, initial laboratory load tests were previously done on the prototype bridge guardrail system employed in the anisotropic-grid timber bridge framing concept. The test set-up was devised to conduct tests similar to those previously conducted by the authors et al. (Pellicane et al. 1990, Gutkowski et al. 1990, 1993) on a conventional timber bridge guardrail specimen. The intent was to allow a comparison of behavior of the prototype alternative and the conventional timber guardrail systems under consistent conditions of the test set-up and method. The prototype system was found to have significantly higher load capacity (Favre 1994, 1996). However, the test set-up was such that direct measurement of some critical internal forces in load path for transmitting guardrail loads into the main superstructure was not possible. Hence, a revised test set up was configured. The key new element in the revised guardrail laboratory test set-up was the direct measurement of the horizontal forces transmitted into each floor beam member. The researchers also conducted exploratory dynamic impact tests on the prototype specimen. These tests are the subject matter of this report.

4. RAMP LOAD TESTS

4.1 Physical Configuration

Figure 2 illustrates the complete laboratory test set-up. A 25-ft. long (7.6 m) full-scale segment of the anisotropic-grid bridge through girder and bridge guardrail system was built in the laboratory, and is referred to herein as "SPECIMEN 3." "SPECIMEN 1" and "SPECIMEN 2," which preceded it, were the subject matter of the prior publications cited above.

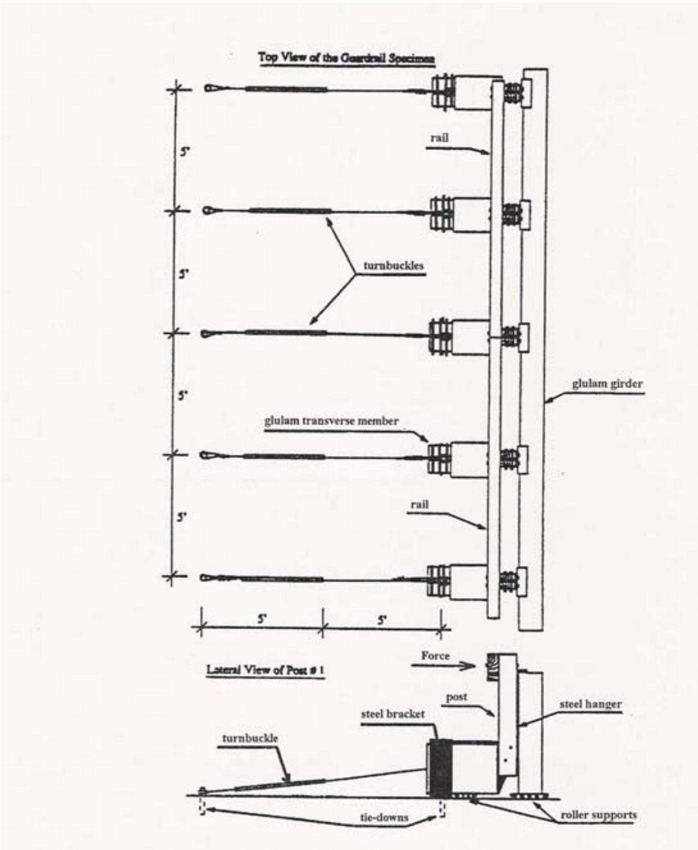

Figure 2 SPECIMEN 3 Laboratory set-up

The cross-section of the glulam through girder was 12 in. x 60 in. (305 mm x 1520 mm). Steel hangers (see Figure 3) were nailed to the side of the through girder (see Figures 4 and 5), with the center-lines of these hangers being spaced 5 ft. (1.524 m) apart along the length. Paired transverse glulam members, with a 2-in. (51 mm) space between them, were attached to the top and bottom plate of each hanger by masonry nails (see Figure 5). Each of the transverse members was 3 x 24 in. (7.6 mm x 610 mm) in cross-section and 3 ft. (0.92 m) long. All timber elements were Douglas Fir Grade #1.

Figure 3 Steel hangers

Figure 4 In-Place hangers

Figure 5 Nailed hanger flange

Each of the five falsework posts was comprised of two nominal size 4 in. x 10 in. x 5 ft. (102 mm x 354 mm x 1.52 m), i.e., actual size 3.5 in. x 9.25 in x 5 ft. (89 mm x 235 mm x 1.52 m), sawn timber members bolted to each of side of the steel hanger web. The longitudal rail was comprised of two nominal size 4 in. x 12 in. (102 mm x 305 mm), i.e., an actual size of 3.375 x 11.5 in. (88 mm x 292 mm), sawn timber members. The rail was attached to each of the five post locations by two 5/8 in. (16 mm) diameter, 14 in. (356 mm) long threaded bolts.

4.2 Specimen Restraint

The laboratory set-up of SPECIMEN 3 was intended to simulate the transverse load path conditions of the actual bridge. A connection system of cables simulated the deck stiffness of the full-size bridge configuration. The horizontal anchoring of the system was done through the transverse members only, with the intent that the entire applied transverse load was transferred into them. When outward transverse loads are applied to the rail, a tendency for an upward rotation of the transverse members will result. In an actual bridge, overall I-beam action minimizes the upward motion. Thus, in setting up the SPECIMEN 3 laboratory, upward motion of the transverse members was restricted. The overall specimen was supported vertically by four roller supports, two at the ends of the main glulam girder and two beneath the outermost pair of transverse members. Each pair of transverse members passed through a steel sleeve (see Figure 6). The steel sleeve was made by welding two vertical 32-in. (810 mm) long C12x30 channels to the ends of a 22 in. (560 mm) long C15x50 steel channel.

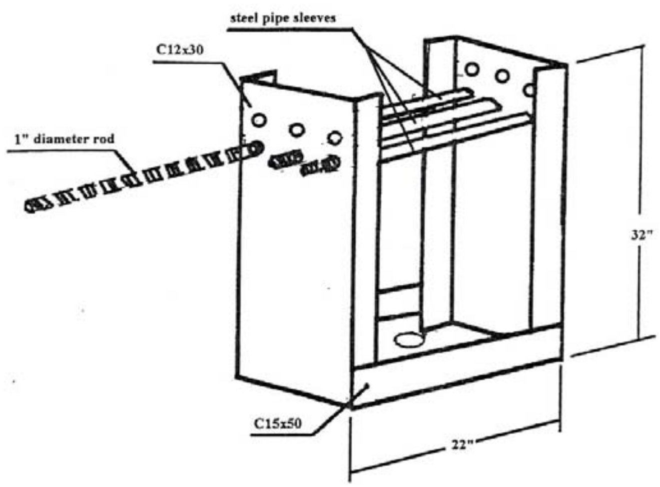

Figure 6 Steel sleeves

The steel sleeves prevented vertical motion of the transverse members while allowing horizontal movement as follows. When each steel sleeve was installed, its horizontal channel was tied down to the concrete floor. The two vertical legs of each sleeve were then joined horizontally by three 1-in. (25 mm) diameter rods as shown in Figure 6. The three rods were passed through steel pipe sleeves, and their ends tightened to the vertical legs of the sleeve. The intent was to allow horizontal motion of the transverse members.

The specimen was lifted until the transverse members were in contact with the steel pipes. It was then held in its elevated position by placing a nominal 2 in. x 4 in. (50 mm x 102 mm) block of wood over each of the roller supports. Prior to placing each steel sleeve, a 1-1/2 in. (292 mm) diameter, 2 ft. (0.61 m) long stud was placed vertically in the 2 in. (51 mm) gap present between each of the paired transverse members. The stud bore against five horizontal 5/8 in. (16 mm) diameter rods placed in predrilled holes through each transverse member (see Figure 7). After placing each steel sleeve, steel cables and a turnbuckle were used to connect the studs to a tie-down in the concrete floor (see Figure 7). This arrangement transmitted most of the horizontal load through the turnbuckles.

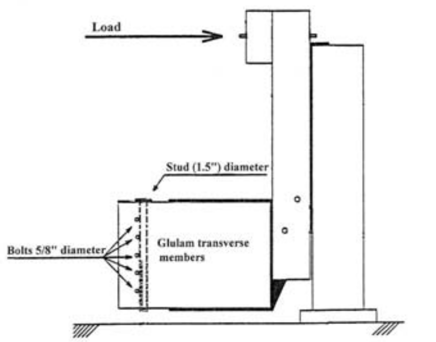

Figure 7 Guardrail transverse beam connection

4.3 Instrumentation

Electrical strain gages were mounted to each turnbuckle (hereafter, termed a “load link”) used to tie the specimen horizontally. These served to measure the horizontal forces in the transverse members. Four 350-ohm strain gages were mounted to one of the eyebolt ends of each load link, where the surface was smoothly sanded (see Figure 8).

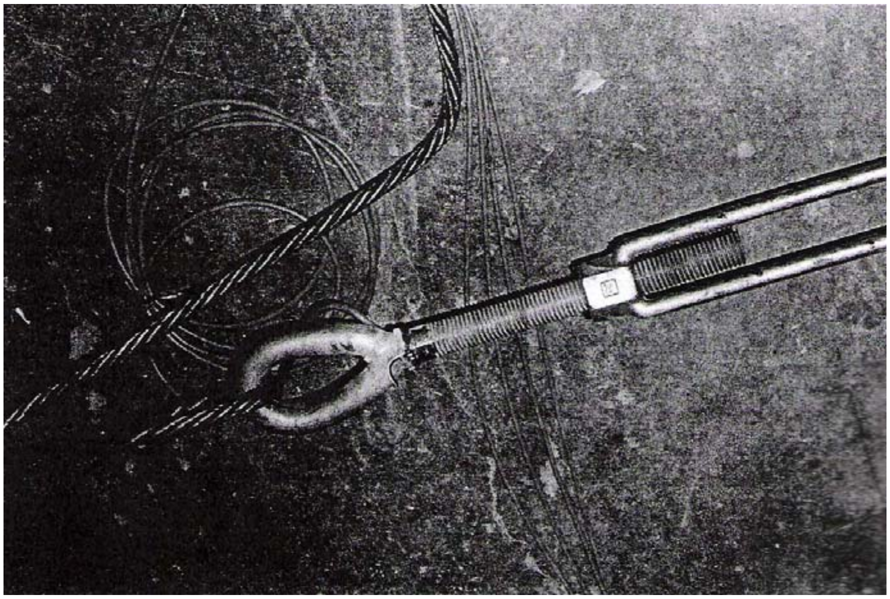

Figure 8 Load link

Each load link was calibrated to establish its linearity and to determine its calibration factor. The Wheatstone bridge of each load link was wired to a strain indicator. The load link was then mounted in an Instron universal test machine and subjected to tensile load increments up to a total load of about 5000 lbs. (22.3 kN). For each of the five load links, the results indicated that the change in strain was directly proportional to the applied tensile force. Subsequently, each load link was calibrated with the winch load cell to determine its load when tied down to the floor. A ramp load up to about 5000 lbs. (22.3 kN) was applied to the load link, with readings of the output voltage taken incrementally. The load links were observed to be linear and accurate to about ±15 lbs. (67 N) at the 5000 lbs. (22.3 kN) level.

Horizontal displacement of the rail at each of the post locations was measured with displacement transducers. The transducers were linear and accurate to about ±0.001 in. (0.025 mm). Each transducer was fixed to its own support system, which was then placed on one of the pairs of the transverse member. Consequently, the rail displacement was measured relative to the transverse members.

4.4 Loading Procedure

Load was applied using a movable winch mounted on a frame system located parallel to the specimens (see Figure 9). The winch cable was placed so as to be able to apply transverse load to the wood rail opposite the middle post of the specimen. A block and tackle device was used to mechanically increase the winch load limit of 40,000 lbs. (178 kN). A steel frame designed to support the cable system and resist its reactions was erected between the winch and the specimen. The magnitude of the applied transverse load was measured by a load cell installed along the winch cable. A stiffened steel plate distributed the load to the wood rail over an area of about 25 in.^{2} (161 cm^2) (see Figure 10).

Starting with the south post location, displacement transducers were numbered from #1 to #5 (Figure 11). The loading device was attached to the rail at the position of the middle post. The specimen was preloaded by subjecting it to a sequence of ramp loading and unloading cycles ranging from 0 to about 36,000 lbs. (160 kN). This served to eliminate the slack between the different components of the specimen set-up. The turnbuckles were then turned to tighten the horizontal steel cables restraining the floor beams. Base readings were taken from the displacement transducers and the load links. The specimen was then ramp loaded up to 32,520 lbs. (1458 kN) and readings of instrumentation were taken at increments of about 4,000 lbs. (0.9 kN). The specimen was then unloaded without reading the instrumentation

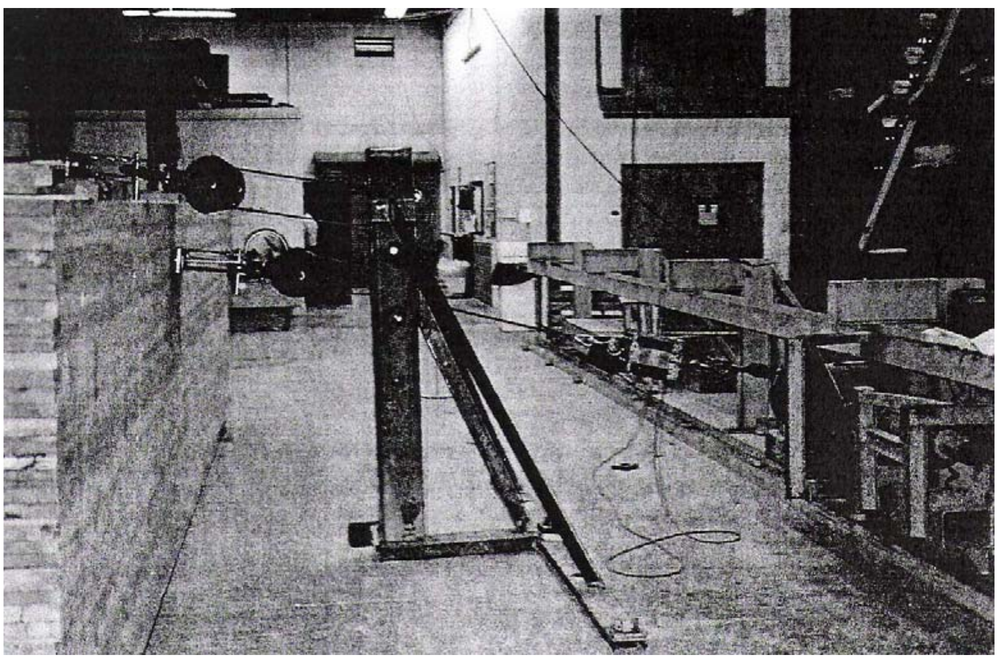

Figure 9 Pulley set-up of the loading system

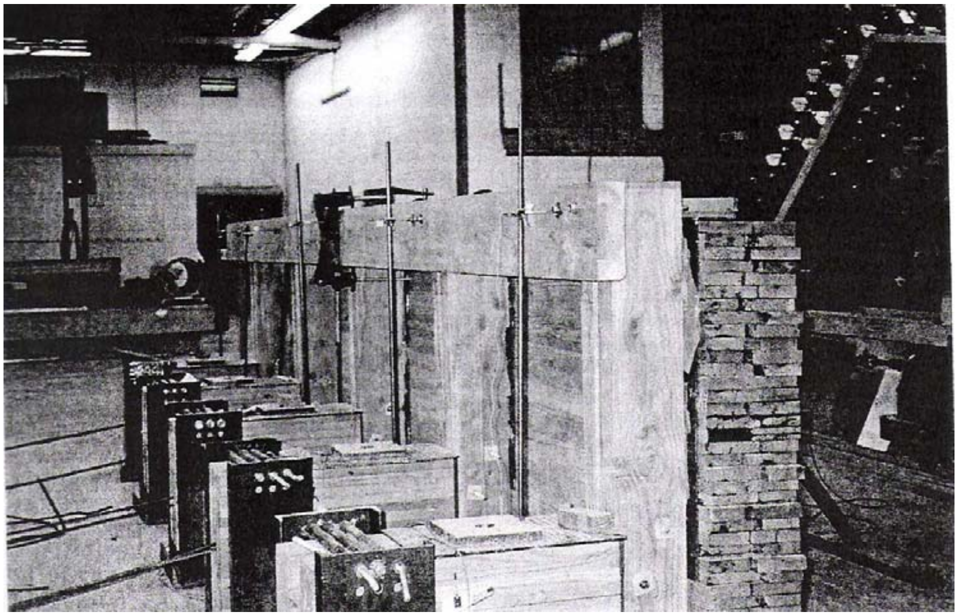

Figure 10 Rail loading fixture

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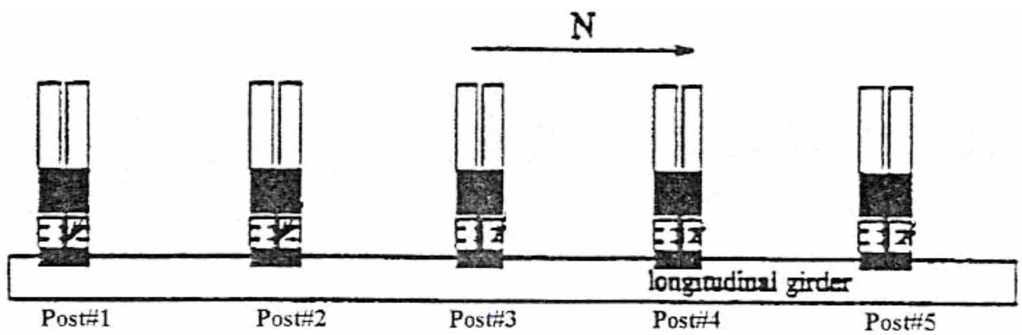

Figure 11 Guardrail post numbering

4.5 Results

Readings obtained from the load links provided the force in each pair of the transverse members. Due to the physical constraints, the cable lines were inclined at an angle of approximately 14 degrees. Also, each of the steel cables transmitting the load from the transverse members to the floor tie downs had a 14-degree angle of depression. Since the angle of elevation of the loading cables was equal to the angle of depression of the restraining cables, the measured values of the loads could be used directly for comparison purposes.

Figure 12 shows the measured forces created in the transverse members due to a horizontal force H applied to the rail at the location of the middle post. The measured resistances to the applied load are not evenly distributed among the five pairs of transverse members. The two outermost pairs resisted the largest share of the applied load. When the sum of the resisting forces transferred to the load links (ΣR) was compared to the applied load H, it was found that the former was measurably less than the latter. Also, the percentage of the load transferred ($\Sigma R/H$ %) decreased with increasing load level. Figure 13 shows this gradual decline in the percentage.

The difference in total applied load and total resistance is attributed to the unintended frictional forces present in the system. The transverse load applied to the rail tended to rotate the specimen about the supports under the through girder. This caused the transverse members to bear upward against the pipes of the steel sleeves, likely partially restraining their rolling action. This appears to have caused significant frictional forces. As the applied load increased, the friction forces would also increase to different degrees. Thus, the percentage of load transferred to each transverse member would change as well. In an attempt to reduce the friction forces, pieces of Teflon were placed between the transverse members and the top chords of the sleeves. This had no noticeable effect on the observed results. The applied load also produced torsion on the through girder. Because of the stiffness of the through girder, the girder soffit displaced in a direction opposite to the applied load and relieved some of the tension forces in the load links. The measured resisting forces include the net effect of these phenomena.

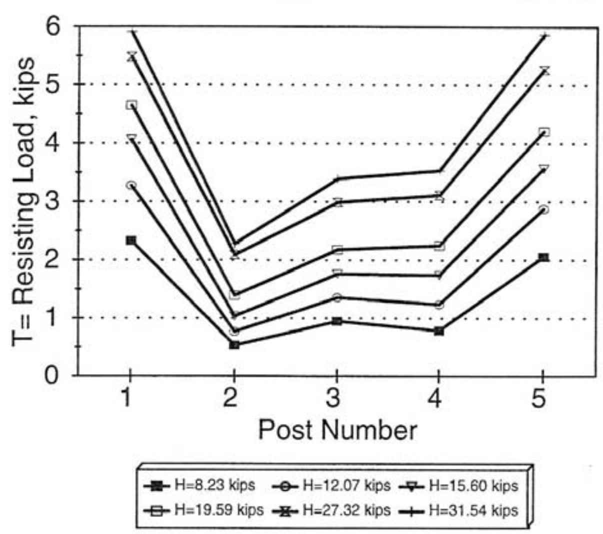

Figure 12 Measured forces in transverse members

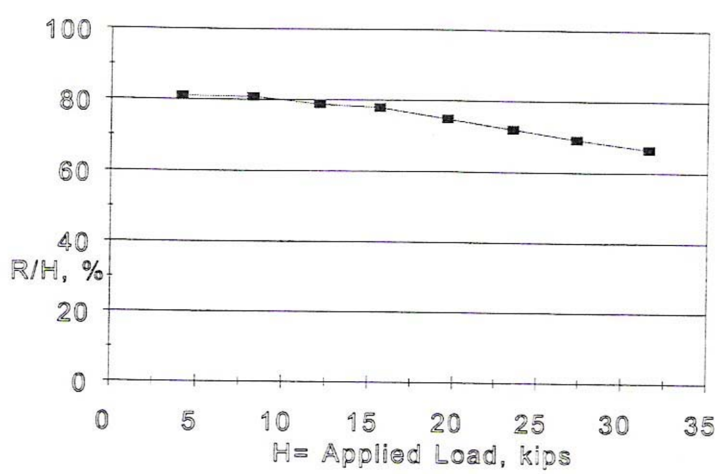

Figure 13 Percentage of total applied load transferred

Bar charts shown in Figure 14 depict the percentage distribution of the total applied and of the loads transferred to the transverse members. A degree of symmetry is evident in the distribution of the transferred loads. The two outermost pairs of members together had to accommodate more than 35% of the applied load and more than 50% of the transferred load. The percentage of the load resisted by the outermost pairs of transverse members decreased with increasing applied loads.

Also, the load shares resisted by members at the locations of posts #2, #3, and #4 increased slightly. The load distribution observed in the transverse members can be explained to a great extent by considering the measured pattern of the rail deflection (Figure 15) relative to the transverse members. For any applied load, post #3 experienced the highest deflection, followed by post #4, then post #2, while the two outer most posts experienced minimal relative deflection.

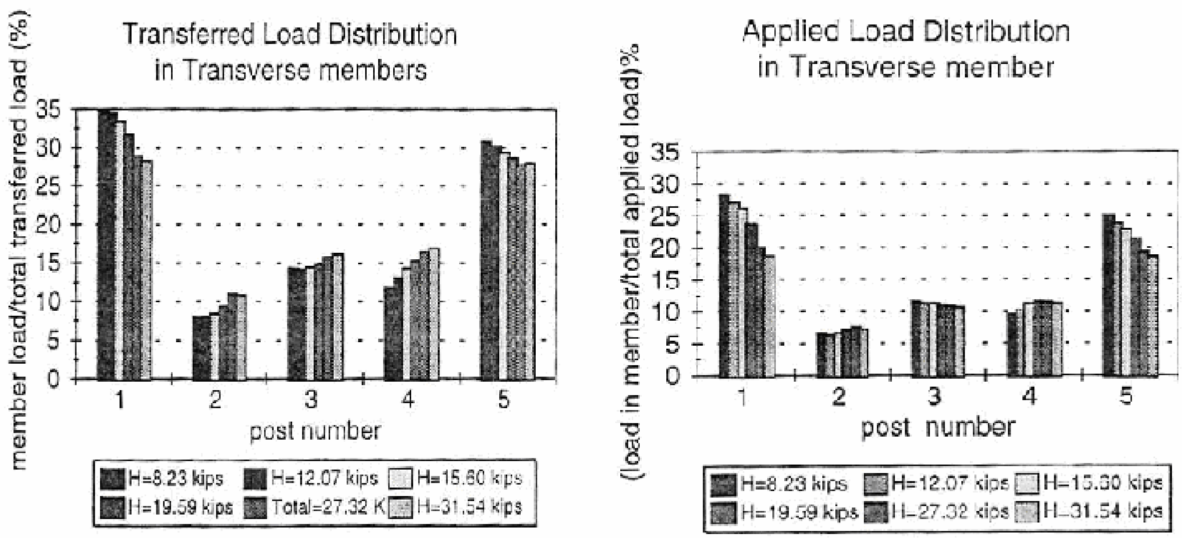

Figure 14 Load distribution in transverse members

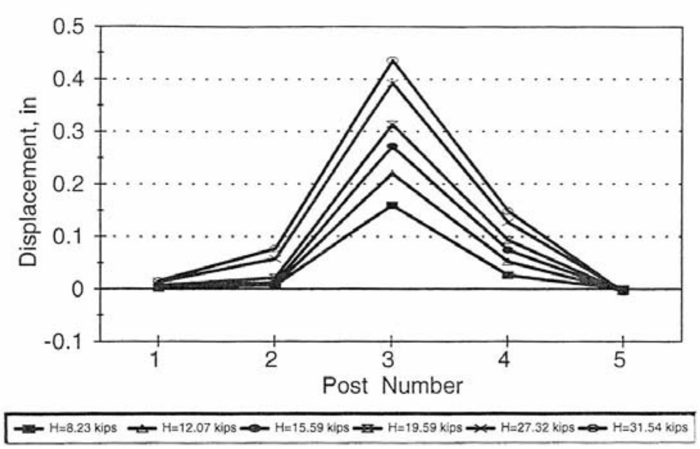

Figure 15 Deflection pattern of guardrail

The pattern of the relative deflections suggests a similar distribution pattern of the magnitude of the upward forces exerted by the transverse members on the steel sleeves. The corresponding friction forces created by these upward forces would be highest at the location of the middle post (post #3), less at posts #2 and #4, respectively, and least at the two outer most posts (posts #1 and #5). The friction forces developed (together with the forces released by the base movement of the through girder) and the horizontal forces in the restraining cables had to equilibrate the applied load. Thus, each pair of transverse members would experience a reduction in the value of the horizontal forces, which would have been transferred to it.

The magnitude of the loss in the horizontal forces going to each pair of transverse members would be equal to the sum of the magnitudes of the horizontal frictional forces experienced by that pair, and the load lost due to the movement of the through girder. However, none of the transverse members resisted more than 35% of the load applied to the rail. The distribution of the applied transverse load in the transverse members differs considerably from the distribution of the forces applied to the posts at the rail level. Based on prior tests of SPECIMEN 2, about 85% of the applied load was transmitted to the loaded post. It is deduced that the load path between the post and the supporting components of the superstructure caused a redistributing of the transverse load applied to the rail into that experienced by the transverse members. Otherwise, the outer transverse members would not have provided such high percentages of the total resisting force. In the absence of frictional effects present in the laboratory specimen, the real bridge would have a more balanced sharing of forces in the transverse members.

5. DYNAMIC IMPACT LOAD TESTS

Under ramp loading, the specimen remained undamaged up to a load of 32,000 lbs. (160 kN), at which time the loading was terminated. Facilities and funds were not available to arrange a federal crash test, so exploratory laboratory dynamic impact tests were then conducted. Initially, several low impact tests were conducted using a pendulum to examine impulse time characteristics. Details of the test set-up are given in a thesis (Shigidi 1995). The impact mass was 385 lbm. (175 kg). Test drops from heights up to 77.25 in. (1.96 m) were used and produced theoretical velocities at impact up to 13.88 mph (6.21 m/s). Actual velocities were not measured and were estimated from the governing formula, $\upsilon=\sqrt{2gh}$, where υ = velocity in ft/s, g = gravitational acceleration = 32.2 ft/sec^2, and h = the drop height. However, a load cell recorded force at the impact location and accelerometers recorded data at a sampling rate of 50,000 Hz.

Figure 16 shows the impact force/time response at the load cell for two drop heights, namely 75.75 in. (1.92 m) and 77.25 in. (1.92 m). The resulting impact velocities were 13.75 mph (6.15 m/s) and 13.88 mph (6.21 m/s), respectively. The impact produced two peaks, which was typically in other drops as well. The first peak occurs when the impacted rail accelerates to the velocity of the pendulum mass. Once the rail starts deflecting and moving with the pendulum mass, the force drops to zero. As the wooden post to which the rail is attached bends, it stiffens and the second peak occurs. Later, small peaks are from the secondary impacts after the pendulum mass rebounds from the initial impact.

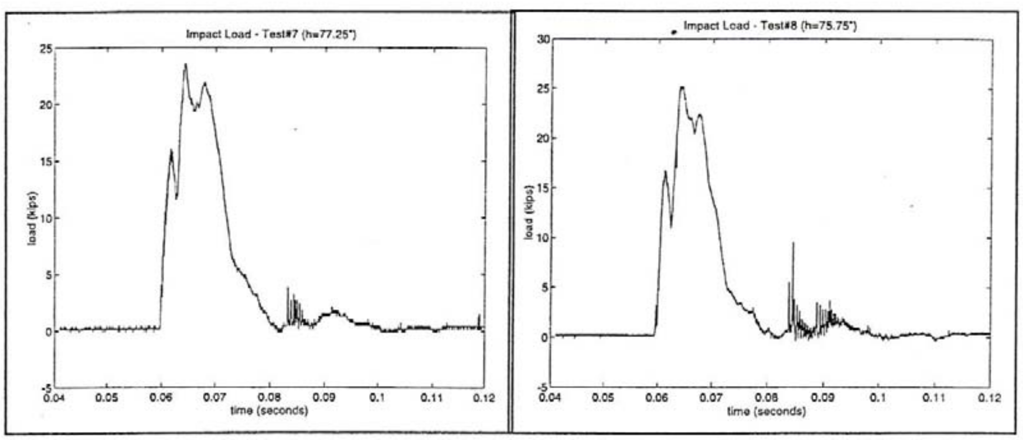

Figure 16 Impact force/time responses

Impulse severity, IS, for a frontal impact is given by

$$IS = 1/2\ mv^2 \qquad (1)$$

in which m is the mass and v is its velocity at impact.

The total impulse generated by the dynamic force, f(t) is given by

$$\text{Impulse} = \int_{t_1}^{t_2} f(t)\,dt \qquad (2)$$

when integrated over the impulse time period from t_1 to t_2. Using the trapezoidal rule, Equation 2 was integrated over the time period of the primary impact comprised of the two main peaks. For the two cases in Figure 16, the impact times were 21.6 msec and 21.4 msec, respectively. These are less than the 0.3 secs. typical of a crash test of a longitudinal barrier or other highway safety feature (Ross 1993). In those tests, crushing of the vehicle front components attenuales the impact and increases the duration of the impact event. The peak impact forces were about 25 kips (111kN) and 24 kips (107 kN), respectively, which are less than the 32 kips (142 kN) applied in the prior ramp load testing. The visual examination of the guardrail after the tests did not reveal any visible failure in the components comprising the specimen.

While not a part of this phase of the project, an additional informal exploratory impact study was done under much heavier impact load, using a much heavier pendulum set up. Due to the tie-down locations and need to resist greater tendency for overturning in the specimen, the segments of transverse members were replaced with longer members. The specimen was then positioned where it could be struck by a large concrete block suspended from an overhead steel frame. The concrete block had a volume of 21.3 $ft.^3$ (87.5 m^3). Using a unit weight of 145 pcf (2320 kg/m^3) its total weight was 3090 lbs. (1400 kg). The leading face of the concrete block was fitted by a crushable fixture (see Figure 17) to simulate the effects of crushing of a vehicle front end during impact, as required by NCHRP guidelines for such tests.

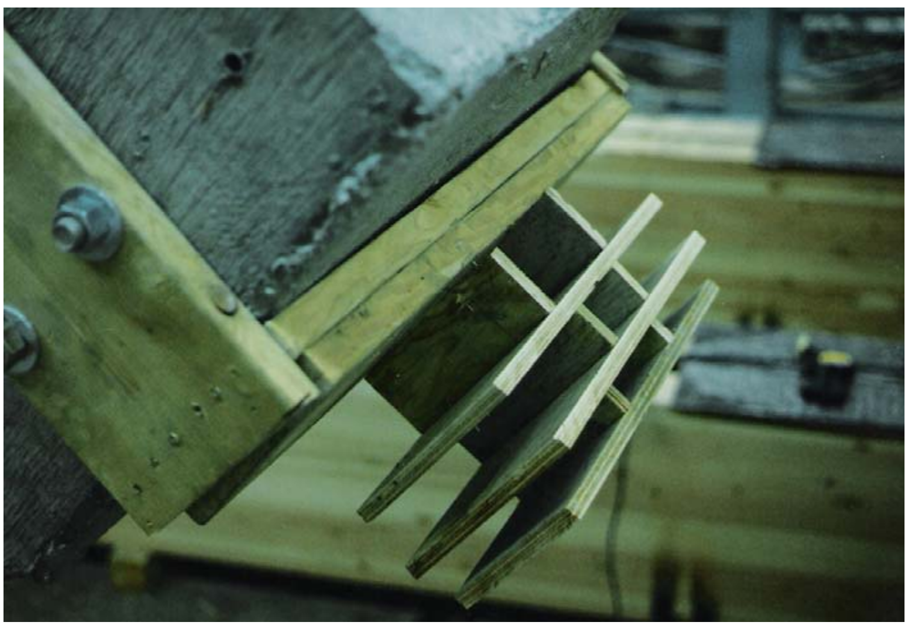

Figure 17 Crushable fixture

Because the load capacity of the original specimen was so much higher (evident from the static load test) than the then required AASHTO static design load factored to the ultimate load level, two significant changes were made. First, the outer side rows of nails were removed from the top and bottom flanges of the steel hanger (see Figure 18). Second, approximately 12 in. (300 mm) of the depth of the main girder of the specimen was sawn off the top of that member (see Figure 19). These changes were intended to be major downgrades in the specimen resistance. Also, as the lever arm of the transverse members was longer, the restraint to overturning of the main girder under lateral load was significantly higher. This led to a much more rigid specimen at impact compared with the original specimen used in the ramp load tests. Presumably, a much higher impact force would result over a shorter impact time than would have occurred for the original specimen, which had been sized in accordance with the geometry intended for an actual bridge application.


Figure 18 Removal of flange nails


Figure 19 Through girder reduced in depth

After positioning the modified specimen into its new location, the pendulum (concrete block) was then raised to the position shown in Figure 20. It was set to swing over impact the rail when the concrete block was horizontal. For safety reasons, an actual initial elevated position was not measured. The center of the concrete block was later estimated (from photographs) to be 4 ft. 1.5 in. (1.26 m) above the impact elevation. Upon release, the block swung downward and impacted the specimen at the rail opposite a post and hanger more or less on center. It then rebounded and hit the specimen again a few times but at substantially reduced speed. The initial impact velocity for the estimated position of the concrete block was 11.1 mph (5.0 m/s). The corresponding impact severity (kinetic energy) is 12.8 kip-ft. (17.4 kN-m).

Figure 20 Pendulum test set-up

The main damage was to the through girder, and was comprised of several discontinuous, perpendicular-to-grain cracks (appearing almost like opened seasoning checks) horizontally at various locations on the impact side of the member (see Figure 21). This is logical as the through girder tended to rotate (torque) outward away from the concrete block upon impact and flex about the essentially rigid base somewhat like a cantilever plate. Moving the specimen into position also involved considerable lifting and rotation of it, as shown in Figures 22 and 23. It is possible this handling created some initial unseen micro-damage prior to the pendulum test itself. However, after the impact of the pendulum, the specimen had no overall catastrophic failure. High speed film also revealed it resisted the impact load more or less like an immovable mass absorbing a shock effect. However, the wood rail did flex oscillate in a wave form for a short time. Had the overall specimen been able to rotate more flexibly, as in a real bridge, the impact duration would have been lengthened in time, and the impact force would have been reduced in magnitude. Thus, the effect of impact effect is considered to be much more intense than in an actual bridge hit by the same magnitude of impact.

Figure 21 Horizontal cracks in through girder

Figure 22 Lifting of specimen

Figure 23 Position of specimen during move to test location

Most of the other physical damage was minor and due to features and components of the test set-up that would not be present in an actual bridge. Primarily, the portion of the steel hanger protruding above the cut main girder was plastically deformed by flexure (see Figure 24). This was an oversight, and the hanger should have been blocked in some way locally at the hanger locations to prevent such unrestrained deformation. In actuality, the hanger would be flush atop the girder. The concrete block left an imprint on and caused some minor breakage into the wood rail, but did not significantly damage it (see Figure 25).

Figure 24 Permanent deformation of protruding hangers

Figure 25 Bearing indentation in rail

The transverse members showed the effect of being vertically tied down at their ends. A perpendicular to grain tension failure occurred at the base some of the members near their ends (see Figure 26).The horizontal thrust applied to the specimen caused a few of the lateral rods used to transfer the load from the transverse members to the steel tie-downs to experience permanent flexural deformation due to the impact, but did not fail (see Figure 27). One of the posts on the back side of the impact point sustained a long vertical crack at approximately its mid-width (see Figure 28). The extension of the post above the cut girder is believed to have contributed to that occurrence. In any event, in the designed system, these vertical posts are only blocking members (falsework) and are not counted upon for structural resistance to the lateral impact. The main purpose for the pair of posts is to cover the steel hanger and help protect it from exposure to moisture. Essentially, the posts transfer applied load locally by bearing from the rail to the top of the posts and from the top of the posts to the top of the hanger and girder. That force is then taken by the steel hanger and girder and transferred downward into the transverse member.

Figure 26 Imprint on rail from pendulum impact

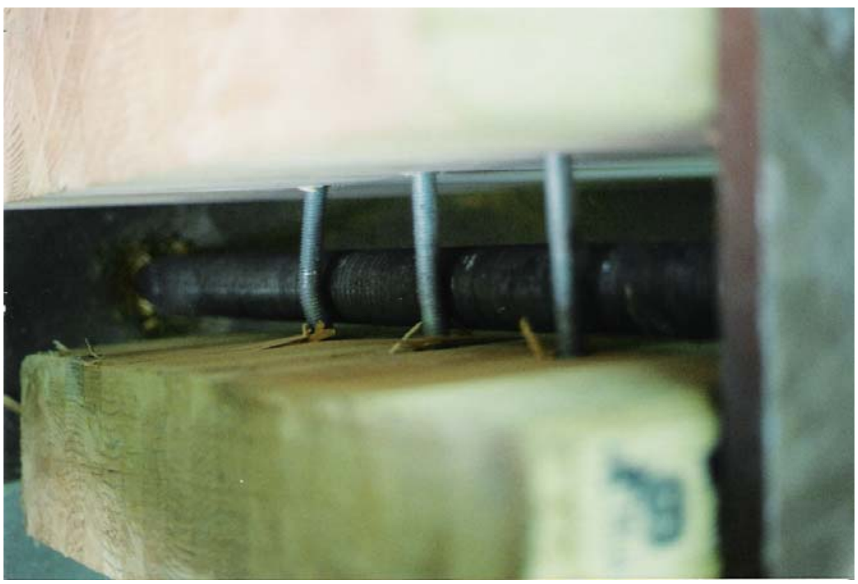

Figure 27 Deformation of steel rods

Figure 28 Vertical split in false work post

6. OBSERVATIONS

The outcomes of the prior laboratory testing at CSU of a then conventional all timber bridge guardrail were reported in past publications (Pellicane et al. 1990, Gutkowski et al. 1994). The alternative bridge guardrail system is a definite improvement in strength. During the ramp load test, the alternative guardrail specimen was loaded up to about 32 kip (142 kN) of transverse load at the center post. Thus it was subjected implies a safety factor of 3.2 compared to the 10 kip (44.5 kN) static design requirement at that time (AASHTO 1989 etc.). This load is 33% higher than the 24 kip (107 kN) load for which the conventional guardrail system completely failed. No visible damage to the specimen occurred. Indeed, since the time of this study, changes and improvements evidently were made independently by others to strengthen the conventional guardrail system (Ritter et al. 1998). Subsequently, the modified conventional guardrail system was successfully subjected to a crash test within the requirements for a then federal Performance Level 1 (PL1) test (Ritter et al. 1998). This suggests the alternative guardrail system might perform successfully if subjected to such a test.

In the ramp load testing, none of the transverse members in the test specimen carried more than 35% of the total axial load transferred to all of them collectively. However, the load distribution observed in the transverse members of the specimen is likely not the same distribution that will develop in the real bridge under similar loading conditions. The laboratory set-up only simulated a portion of the real bridge located well into the middle of the span. Although intended to allow the specimen to move horizontally, the supports and sleeves introduced unintended friction forces in the direction opposite to that motion. On the other hand, the support conditions of a real bridge surely provide even higher countering forces. Thus, the 35% maximum load share is believed to be a high estimate of the maximum load share transferred to a single transverse member, i.e. to a single hanger, and is very conservative.

The exploratory pendulum test with the massive concrete block, suggests good performance of the specimen The impact severity was about 12.8 kip-ft. (17.4 kN-m). This is comparable to a test Level 2 requirement (13.3 kip-ft. [18.0 kN-m]) if conducted with a small sedan 1807 (820 kg) based on federal recommended test procedures (Transportation Research Board 1993). Impact severities required for a 4400-lb. (2000 kg) single unit truck for Test Levels 1, 2, and 3 are 25.4 kip-ft. (34.5 kN-m), 49.9 kip-ft. (67.6 kN-m), and 102 kip-ft. (138 kN-m) respectively.

7. CONCLUSIONS

The overall ramp and dynamic load testing showed encouraging performance of the hanger detail and the special steel connector attaching the through girders to the floor transverse beams. No significant distress in either the steel connector itself or in the nails between the connector and the timber members was observed. The main girder had no visible damage. Loads applied significantly exceeded those that had caused major failure in the conventional guardrail systems when subjected to the same test.

The performance of the test specimens under the concrete block pendulum test is inconclusive. Impact severity achieved was very low, comparable to a small sedan in an NCHRP Test Level 2 setting. Impact severity levels required for a single unit truck are substantially higher. However, the performance in a laboratory test under ramp loading substantially exceeded that of the conventional guardrail system also tested in that manner. When modified, the latter survived a federal PL-1 test concluded by other researchers. These events suggest the test specimen likely had damage to the main girder during its repositioning, thus compromising its capacity. The main girder had some horizontal cracking occur, but extreme handling procedures used to put the specimen in place may have contributed to that. In any event, catastrophic failure events did not occur.

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