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ULTIMATE CAPACITY AND MODE OF FAILURE OF A TIMBER BRIDGE GUARDRAIL

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by

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EXECUTIVE SUMMARY

This report is an interim report for the first phase of an ongoing project entitled "Moderate Weight Bridge Guardrail for Rural Sites." The objective is to examine the potential of a new bridge guardrail concept as a means of increasing the safety of travel over rural bridges.

Numerous guardrails on low volume, rural bridges are unsafe in configuration and connection detailing. Current standard guardrails are substantial in size and costly. In many cases, due to low base budgets for bridges, local decisions to use either a nominal or no guardrail have resulted when Federal funds are not involved. A properly designed, moderate weight system is possible, as determined by computation in a previous MPC project. This project is being conducted to examine the structural resistance and strength of the moderate weight alternative.

Full-scale exploratory laboratory tests will be conducted on a moderate weight guardrail system for potential use on rural bridges. To date, load tests have been completed on another (much heavier) guardrail system that previously passed an optional federal crash test. Tests results on the alternative system will be compared with those results. This will serve as an indicator of acceptability of the moderate weight concept and its likelihood of surviving a crash test.

Load tests were conducted on a full-scale specimen of a section of all-timber bridge, including the guardrail system. The guardrail configuration used was that of the system that had been successfully subjected to a federal crash test. At the time, it was the only all timber guardrail system that had been so tested. It serves as a control specimen for research on alternative systems. The research described herein is being followed by the conduct of similar tests on an improved guardrail system.

A 25-foot-long section of a longitudinal deck bridge with the guardrail posts included (without a rail attached) was constructed. A series of static load tests were conducted on individual posts. Transverse load was applied to the posts and increased to failure levels. The outcomes enabled improvements in the test procedure and modifications to the specimen to prevent premature local failures.

Subsequently, a bridge specimen with five posts and a rail attached was tested under transverse loading. Load was increased until failure, with load and displacement monitored electronically. Failure occurred at a load of 24,000+ lbs. Compared to the AASHTO design load level of 10,000 lbs, this represents a factor of safety of about 2.4. This is considered to be a rational, expected margin for the specimen.

This report presents the detailed conduct and findings tests to destruction performed on the control specimen. Results of tests on the alternative system will be presented in an interim report for the second phase of the project.

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CHAPTER I

INTRODUCTION

Background

Rural Roads and Bridges in the U.S.

Due to a concentrated effort spent on developing and improving the Interstate System, most of the U.S. Rural bridges are of pre-1940 vintage, and thus have exceeded their service life.

A report from the American Association of State Highway and Transportation Officials (AASHTO) evaluated the annual cost of maintaining existing highways and bridges to about $80 billion until the year 2020 [American Association of State Highway and Transportation Officials, Inc., 1988]. The report indicates that of the 571,308 U.S. bridges over 20 feet in length, 371,041 (65 percent) are located in rural areas.

The same report, using data of a National Bridge Inventory developed and maintained by the Federal Highway Administration (FHWA), indicated that 131,248 bridges are currently structurally and/or functionally deficient and another 141,857 bridges are accruing to be deficient by the year 2020. The FHWA's Eighth Report to Congress [Federal Highway Administration, 1987] gave an even higher number of 220,000 currently deficient bridges.

Presently, a serious concern about rural transportation facilities exists amongst government agencies. Efforts are ongoing for the improvement of all types of existing bridges, regardless of material used and span to be bridged. The Federal Highway Bridge Repair and Replacement Program (HBRRP) is making funds available and about 2,000 - 5,000 bridges are removed from the National Bridge Inventory each year.

Timber Bridges

Approximately 71,200 (14 percent) highway bridges use timber as the main material. Additionally 8,500 timber bridges are part of the national forest system and the railway companies use about 1,800 miles of timber bridges [Gutkowski, 1984].

As the HBRRP only distributes between 20 percent and 35 percent of its funds to secondary roads, about 62 percent of local highway officials report inadequate funds for bridges. Thus a National Timber Bridge Initiative has focused on trying to demonstrate cost efficient timber bridge constructions. Prior to this initiative, the USDA - Forest Service and the American Institute of Timber Construction developed a Timber Bridge Technology Transfer Plan (TBTTP). This plan, developed in cooperation with government and numerous private organizations, had the goals of increasing the use of timber bridges by spreading new technologies, organizing workshops and topics which need more investigation to better design new bridges.

Numerous guardrails on low volume, rural bridges are unsafe in configuration and connection detailing. In 1983, participants in a national workshop prepared recommendations for priority structural wood research needs in eight broad topic areas [Gutkowski, 1983]. Participants in a session on Heavy Timber Structures and Bridges identified one of their six primary design needs as "methods for the design of timber guardrails in accordance with the American Association of State Highway and Transportation Officials criteria".

Objectives

Current standard timber guardrails are substantial in size and costly. Colorado State University (CSU) has been conducting a program of research to study the load-displacement behavior of bridge guardrails. Two master of science theses [Malone, 1987; Kälin, 1989] and two research reports by Malone, et al., 1987 and Kälin, et al., 1989 present the initial results of the development of a semi-empirical static load model. A series of static load tests were conducted on a particular five-post guardrail specimen. The results were used to validate the derived mathematical model under nondestructive load levels. This model uses the stiffness matrix method, with theoretically and experimentally derived stiffness matrices, to represent the guardrail specimen. However, these studies were limited to low load levels and did not address ultimate load capacity of the specimen.

Although code specifications exist for the analytical design of bridge guardrails, the situation is changing. Optional federal performance test standards exist for bridge guardrails. In order to satisfy these federal safety requirements, costly crash testing is necessary. By experimentally studying the ultimate static load capacity of a full-scale guardrail specimen, it is possible to pretest new guardrail systems in advance of a decision to proceed to a crash test.

In a recent project on "Efficient Rural Bridges Technologies", CSU researchers developed experimental bridges intended for implementation on rural roads. Funding was provided by the Mountain Plains Consortium (MPC). The research focused on computer modeling of new concepts for superstructures, a laboratory test program on materials, models and full-scale load testing. As a proof of concept stage, experimental bridges were constructed at actual sites in Colorado. Working in conjunction with local engineers, a comprehensive set of load tests were conducted in the field. These served to confirm expected behavior of the test bridge and provide a basis for consideration of modifications and improvements.

The preceding MPC project focused on the superstructure design, i.e. the main vertical load carrying system. Attachments, including the bridge guardrail, were not a direct objective of the experimental studies. However, a novel guardrail system that provided the capacity to resist AASHTO required guardrail design loads evolved as an inherent feature of one experimental bridge. An important aspect is that the guardrail principally was configured in a way that it contributed significantly to the vertical load resistance of the superstructure. A byproduct was an inherently high level of resistance to vehicle impact loads, i.e. the loading that bridge guardrails are intended to resist.

Despite the advantages of the experimental guardrail system, the researchers lacked test data to demonstrate its capacity to resist the AASHTO guardrail design loadings. Due to reluctance of the bridge owner to use the experimental system, modifications were made to make it a more conventional system. To enable future acceptance of the intended experimental guardrail system, test data on ultimate load capacity were needed. Consequently, a follow-up research study was funded by the MPC and undertaken by CSU to address this need. This report present the results of the first phase of that study. In this phase, a conventional bridge guardrail system which had been successfully subjected to a federal crash test was tested for ultimate capacity under static load. In the second phase, the alternative experimental guardrail system will be similarly tested for ultimate capacity. Significantly higher load capacity is expected.

The approach taken in the research reported herein was to conduct a laboratory test of short section of a guardrail system. A five-post section was built and loaded to determine its characteristics and ultimate capacity. This approach is useful, in general, to determine the behavior of any bridge guardrail systems and assess if it has a chance to pass a costly federal crash test.

CHAPTER 2

DESCRIPTION OF THE GUARDRAIL SPECIMEN AND LOADING FACILITIES

Guardrail Specimen

The guardrail used for this research was the same specimen used by Malone and Kälin in their past studies. It was composed of a 26 foot long section of a longitudinal laminated deck bridge complete with curb and rail system (Figure 2.1). It was selected because at the time it was the only timber system that has been successfully tested in a full-scale crash test.

The wood material composing each element of the structure was solid sawn Douglas-Fir Larch, Grade #1, creosote-treated. The elements of the structure were connected together as follows:

Deck: 16 nominal 4" x 12" laminates were nailed together by three-quarter inch diameter spikes. Additionally, several Dywidag rods, one-and a-quarter-inch diameter, were used to prestress the deck transversely.

Curbs and Scuppers: Member sizes and connector locations are shown in Figure 2.1. The curbs and scuppers were bolted to the deck by three-quarter-inch diameter bolts with four-inch diameter split ring connectors placed between curb and scupper, and between scupper and deck.

Posts: Five 8" x 12" posts were used at an average spacing of 5'- 8". Each post was attached to the curb by one-and-a-quarter inch diameter bolts and into the deck by 5/8 inch diameter spikes.

Rail: A 5-3/4" x 11" member was connected to the posts by half-inch diameter bolts, two bolts at each posts.

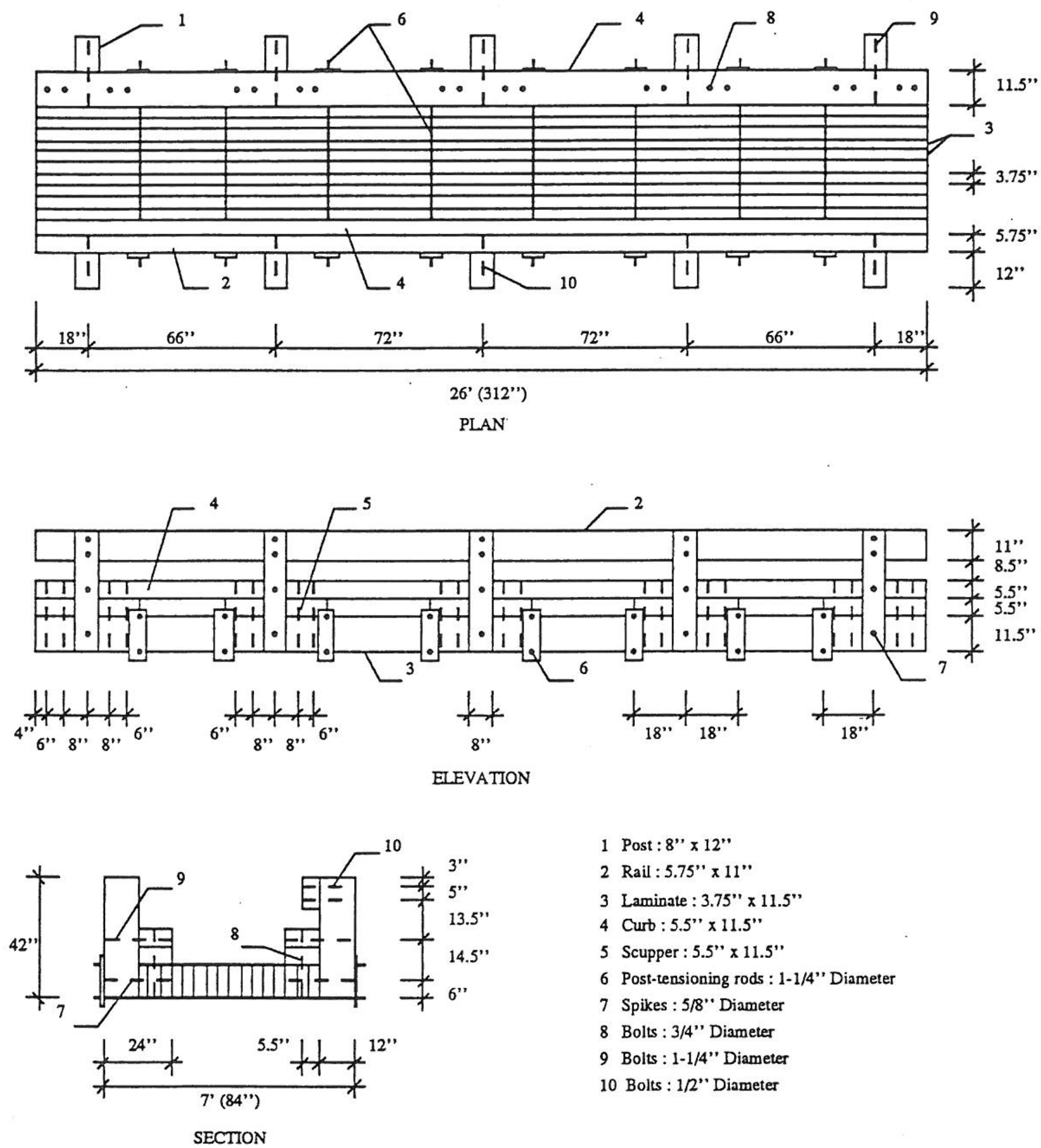

Figure 2.1: Timber Bridge Specimen

Figure 2.2 shows the deck configuration. "A" and "B" identify the alternate pattern of the spikes used in the laminates to help create a continuous longitudinal wood deck. Locations "A" are 14-inch long spikes throughout the width of the deck. Locations "B" use 14-inch long spikes in the interior of the deck with additional 6-inch long spikes to connect the two outermost laminates.

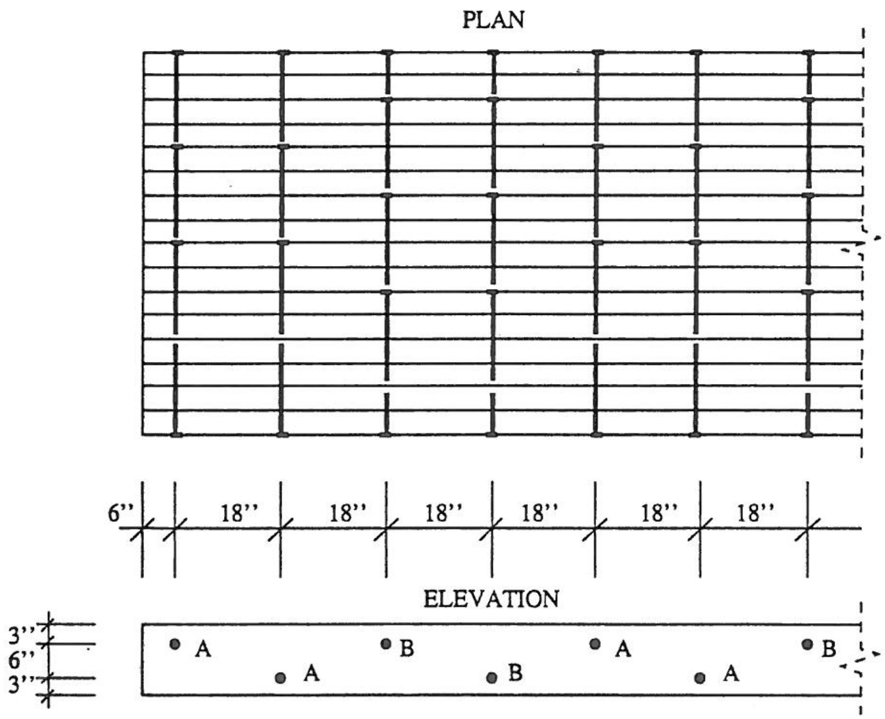

Figure 2.2. Spike Connection Pattern within the Deck

Dywidag prestressing rods were present as a result of the loading tests performed by Kälin, 1989. Initially, Kälin tested the specimen without these rods being present. Subsequently, they were added to simulate a prestressing method then being considered for use in longitudinal deck bridge construction. The prestressing method was developed in 1976 by the Ontario Ministry of Transportation and Communication, 1983. Figure 2.1 shows the location of the rods and Figure 2.3 the prestressing system used for the deck rehabilitation. In the present work, the

Dywidag rods were only tightened manually with wrenches until the gaps between the two extreme laminates and the body of the deck were closed; further tightening was difficult.

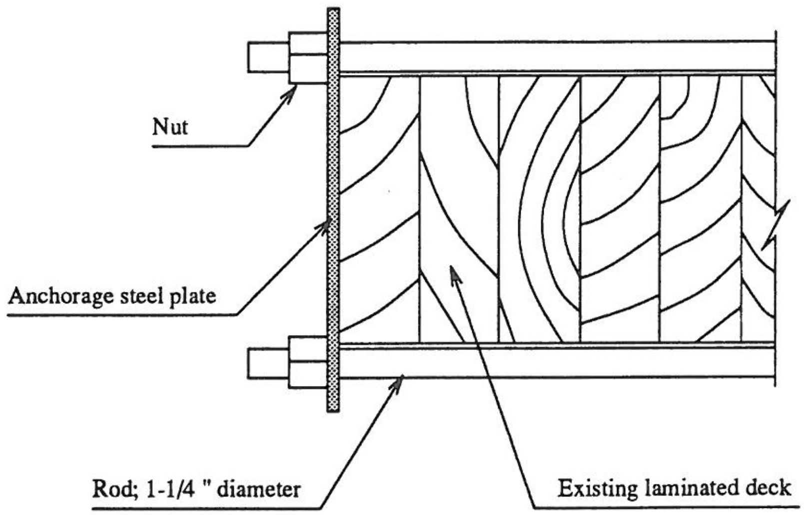

Figure 2.3. Prestressing System for Rehabilitation

One side of the deck was left complete with curb, scupper, posts and rail. On the other side, the rail was removed to allow testing for the ultimate tensile strength of single posts. The tie-down of the specimen to the floor is described subsequently for each set of tests with or without the rail.

Loading Facilities

Testing was conducted in the Structural Engineering Laboratory at the Engineering Research Center at CSU. The laboratory is equipped with a specialized electronically driven winch system used in the testing. The winch is mounted on a rail system and is movable to any position along a 60-foot long supporting structure (Figure 2.4). The load applied by the winch on the different elements of the guardrail system was measured by a load cell installed on the winch cable. The load cell capacity is limited to 10 kips but this can be doubled, quadrupled, etc., by use of block and tackle between the cable and the specimen.

To match the capacity of the loading and measurement devices to the pulling force to break the guardrail structure, two different system were used to apply the needed load. Figure 2.5 shows the simple connection of the cable to a post for load below 10,000 pounds. Figure 2.6 shows the pulley system used when the load at the guardrail structure was under 40,000 pounds. The detail of the connection between the guardrail elements and the pulling system is described (and illustrated) later in the report.

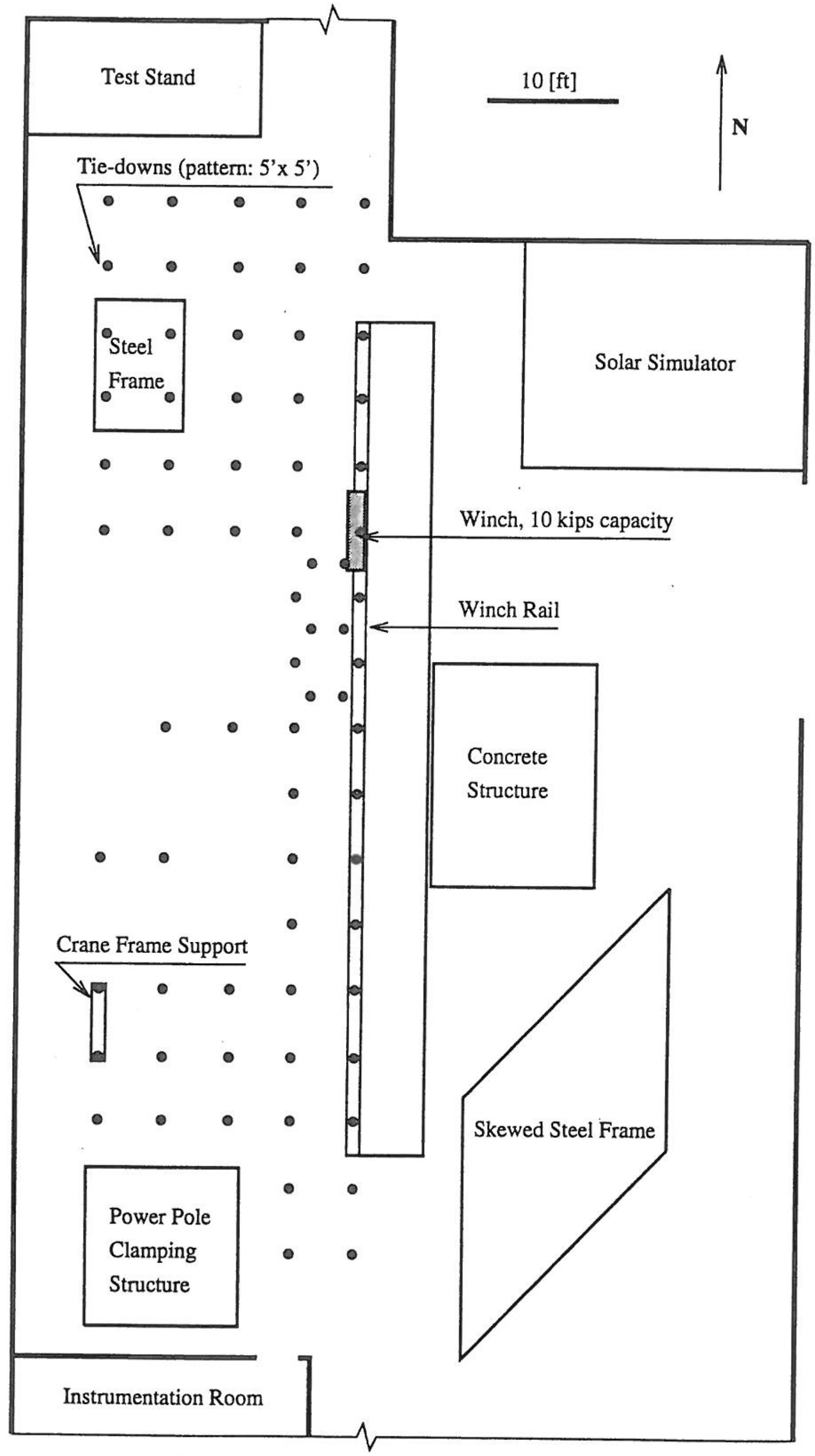

Figure 2.4: Plan of the Laboratory

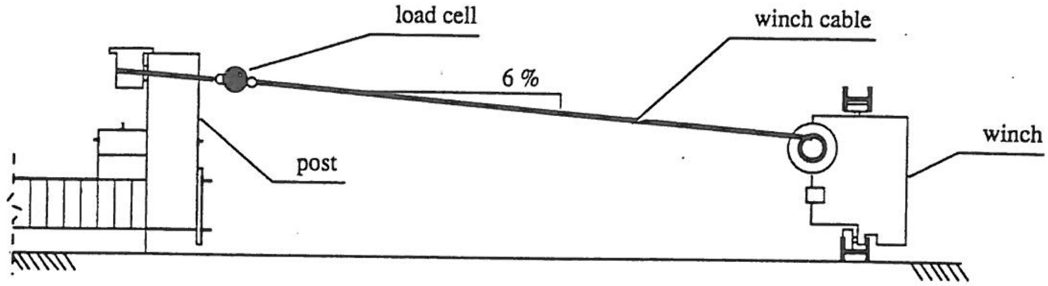

Figure 2.5: Loading System under 10 kips

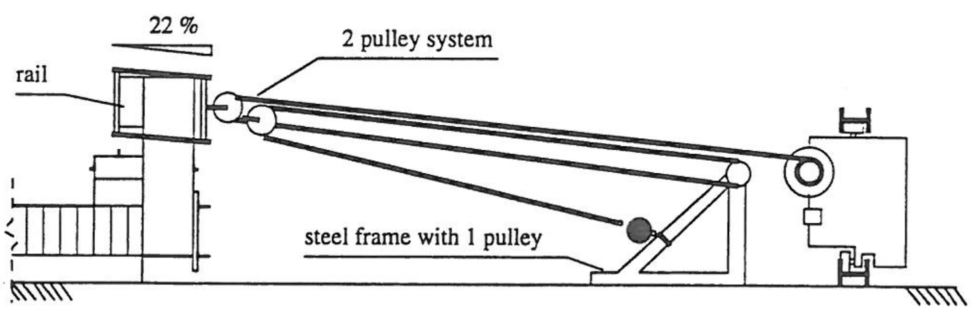

Figure 2.6: Loading System under 40 kips

CHAPTER 3

TESTING OF INDIVIDUAL POSTS

Three separate tests were performed on posts on the side of the bridge specimen that did not have the rail attached to it. One goal was to determine the ultimate capacity of this part of the structure when pulling outward with a static transverse load. A second objective was to determine its mode of failure and the possibility to increase its capacity by modifying the system.

As each test influenced the preparation of the next one, they are successively presented in this chapter after the description of the common set-up of the three tests. Figure 3.1 defines the order in which the posts were loaded.

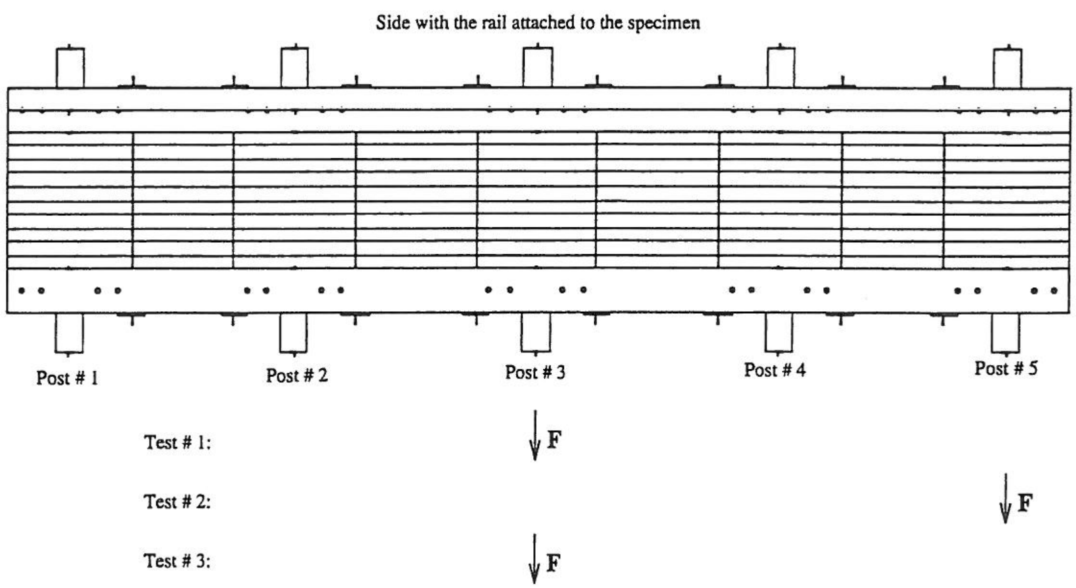

Figure 3.1. Numbering of the POSTS and TESTS

Connection of the Guardrail Specimen to the Floor

The specimen was supported by two pile-caps, one at each end of the longitudinal deck. These pile-caps served to simulate abutments by raising the bridge specimen off the ground. This also prevented the bottom of the posts from hitting the laboratory floor when displacing downward under outward loading.

The tie-down of the deck to the floor was composed of two connection systems. Each wood pile-cap (11" x 6" x 72") was tied down with two 2-inch diameter bolts, spaced 5-feet apart, using existing holes in the laboratory concrete floor. The bolts were recessed to prevent contact with the specimen itself. The bridge deck was connected to each cap with two three-quarter-inch diameter lag screws. The positioning of the lag screws and bolts are represented in Figure 3.2. The 18-inch long lag screws were driven through the entire thickness of the pile-caps.

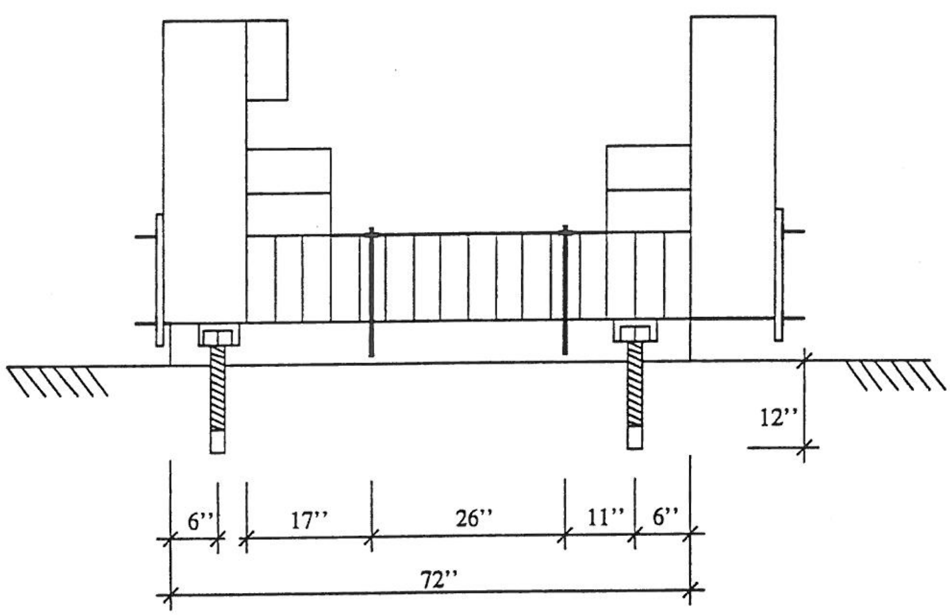

Figure 3.2. Section of the Bridge Specimen

Connection of the Loading Device to the Posts

The connection between the post and the pulling cable system described in Figure 2.5 was the same for the three tests on individual posts. A high load capacity cable surrounded the tested post. The load cell, set into a wood/steel protective envelope, is placed between the winch cable and a high load capacity cable surrounding the tested post (Figure 3.3). To avoid any damage to the post by the bearing force of the high load capacity cable under tension, an eight-inch diameter steel pipe was placed between the post and the cable. From the pipe, the load was transferred to the post by a 16 in^2 steel plate (one-quarter-inch thick). The center of the plate was positioned 5.5 inches down from the top of the post, which corresponded to the centerline of the removed rail. Its purpose was to distribute the load on an area small enough to simulate a concentrated load but large enough to avoid any crushing failure to the post.

TEST #1 Performed on POST #3

Objective

The objective of this first test was two-fold. First, it was desirable to study the response of the guard-rail system (with the rail removed) under a succession of loading and unloading cycles; the magnitude of the hystereses being from 0 to 3,000 pounds. Secondly, it was the intent to load the post until 10,000 pounds, which corresponded to the limit of the loading system in this set-up (Figure 2.5), or until damage occurred in the guardrail/deck specimen.

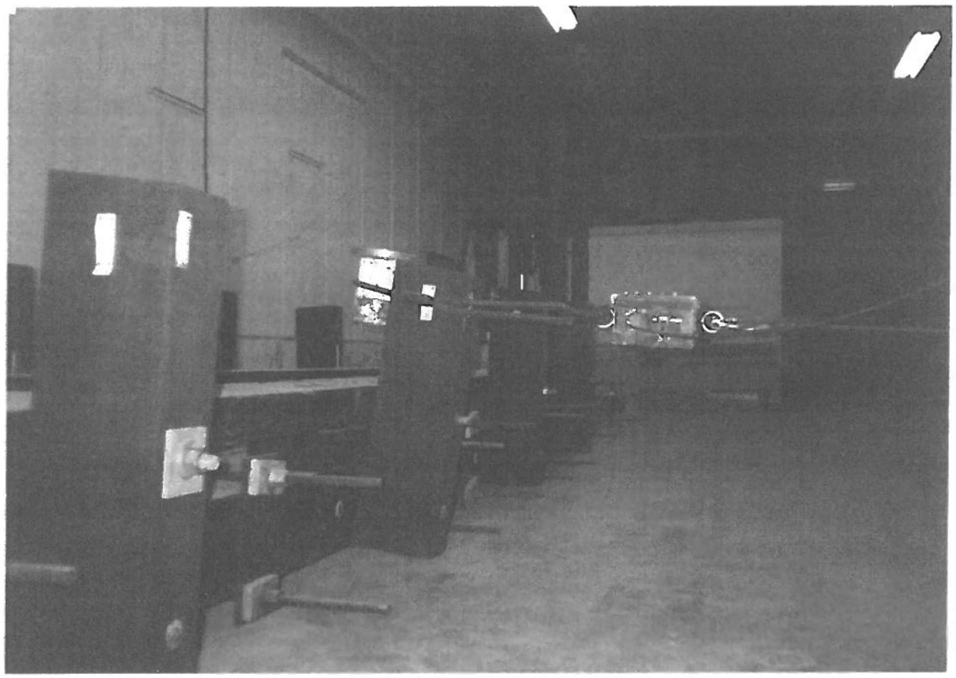

Figure 3.3. Picture of the Loading Device Around the Post

Instrumentation for Displacement Measurement

To collect data of the outward motion of the post at the location of the applied force, a Linear Variable Impedance Transformer (LVIT) was fixed to a support system standing on the laboratory floor (Figure 3.4). The sensor outputs a voltage for any position of its plunger rod. The difference of voltage is proportional to the plunger displacement.

This measurement device was chosen because of its availability, its accuracy (measurement to less than 1/100 of an inch, determined after calibration) and its ability to collect data when the post is returning to its initial position during unloading.

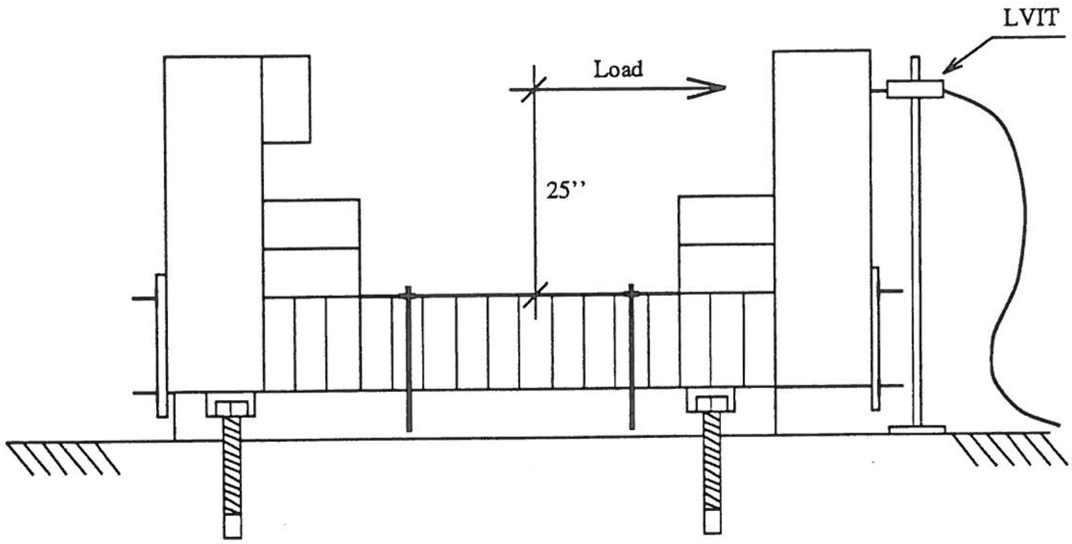

Figure 3.4. Location of the LVIT

Results

Appendix A contains all the data collected during the three tests and used to plot the following graphs. The graphs of Figure 3.5 show the series of hystereses obtained during this test. After the first hystereses (Figure 3.5a), the permanent displacement of the post was 0.25 inch. As shown in Figure 3.7, this outward displacement was measured by the LVIT, 25 inches above the bridge deck. This normal residual displacement can be attributed to the crushing of some wood fibers and the adjustment of the steel/wood connection within the system being tested. With succeeding cycles of load, shown in Figure 3.5b, c and d, the residual displacement was reduced to zero.

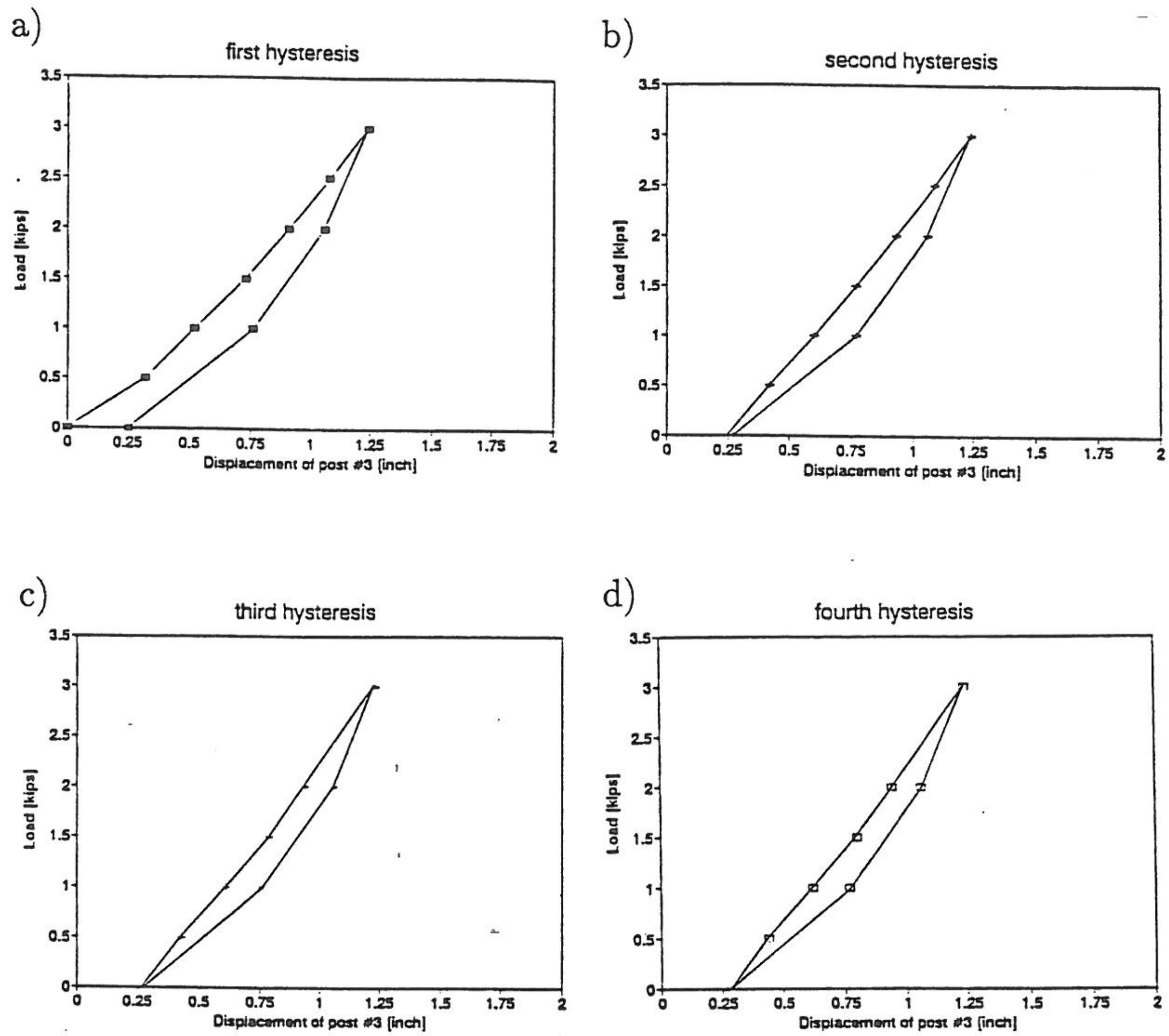

Figure 3.5. Individual Hystereses of POST #3

Figure 3.6 shows the superposition of the four hystereses and the next loading experiment up to 6,000 pounds. The set of hystereses show that the system reacted well under the load magnitude from zero to three thousand pounds. There is no significant residual deformation added after each cycle of loading/unloading. Therefore the loading of the post could be repeated at will and the behavior of the post/deck connections predicted with confidence.

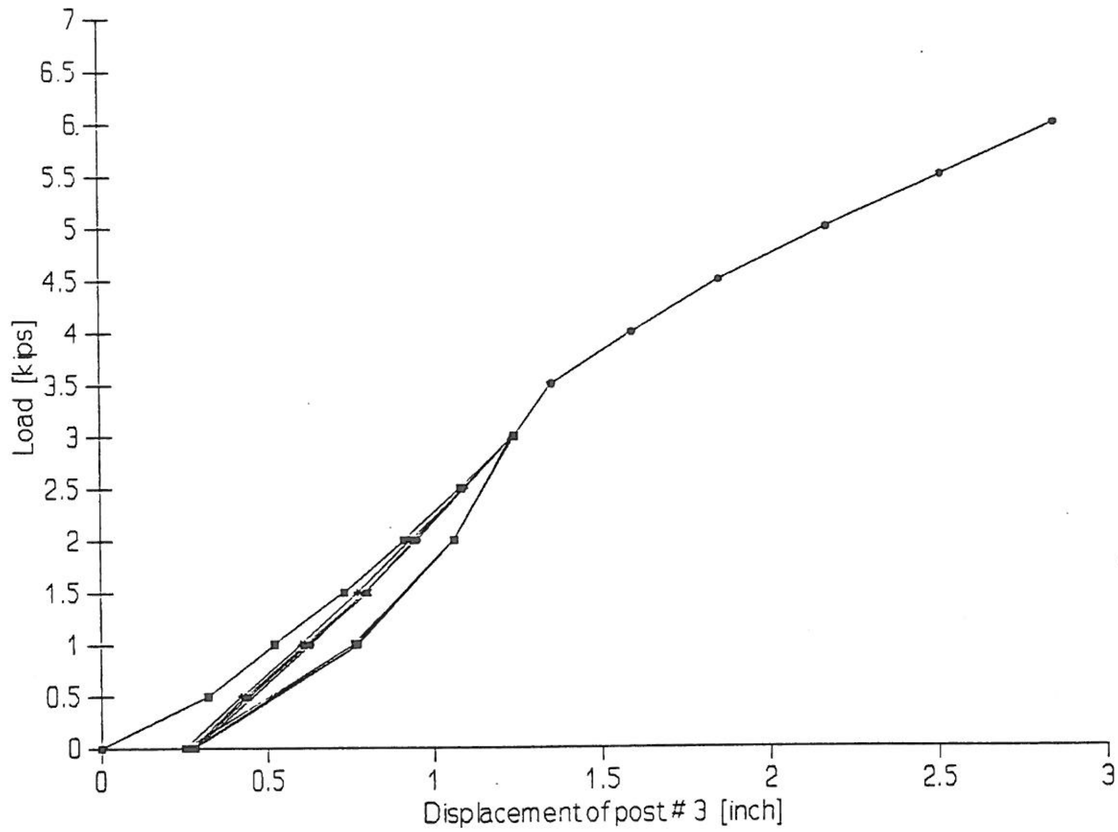

Figure 3.6. Hystereses Superposition of POST #3

Referring to Figure 3.6, a change in the loading response of the middle post occurred between 3,500 and 4,000 pounds. The displacement Δ of the post was higher for each increment of load **P** from this point up. The slope of the load-displacement graph changes from approximately 2.6 kips/inch (within the 0 to 3 kips load range) to approximately 1.7 kips/inch (within the 3.5 to 6 kips load range). This change is attributed to the behavior of the connection of the post to the curb. It was observed that bearing failure caused the nut of the 1-1/4 inch diameter bolt to be pulled through the post. This phenomenon developed gradually and loosened the structure, as shown in the graph.

The loading of POST #3 was terminated at 6,000 pounds with the second stage of damage occurring in the bridge specimen. At this load magnitude, the entire specimen began to rotate out of its position, as shown in exaggerated scale in Figure 3.7. The overturning moment, created by the load being applied 25 inches above the bridge deck, resulted in a moment acting on the abutments. As the upward force increased, the three-quarter-inch diameter spikes screwed in the pile-cap experienced withdrawal and the entire specimen was about to be pulled out of its supports.

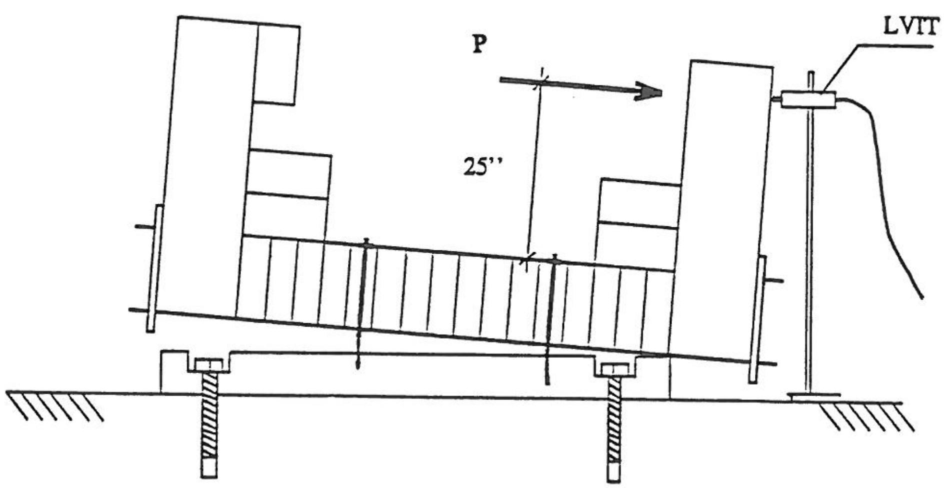

Figure 3.7. Rotation of the Deck

TEST #2 Performed on POST #5

Objective

The objectives of this test were to confirm the need of an improvement for the curb/post connector and to study the behavior of a loaded post at the end of the guardrail specimen.

Displacement Measurement Instrumentation

The instrumentation set-up used for TEST #1 was repeated. It was described in Section 3.3.2.

Results

Three cycles of loading/unloading were used to examine the hysteresis characteristics of the specimen. In each cycle, the range of loading was from 0 to 3,000 pounds. The data for the individual load cycles are detailed in Appendix A. The graph in Figure 3.8 shows the superposition of the three load cycles and the load-displacement response of the post for a subsequent loading up to 7,500 pounds.

The residual displacement of the post after completion of the first load cycle was 0.23 inch. The result is very close to the 0.25 inch observed in cycle one of the load testing of POST #3. The successive hystereses have nearly the same shape and values throughout the experiment, and also when compared to the test of POST #3.

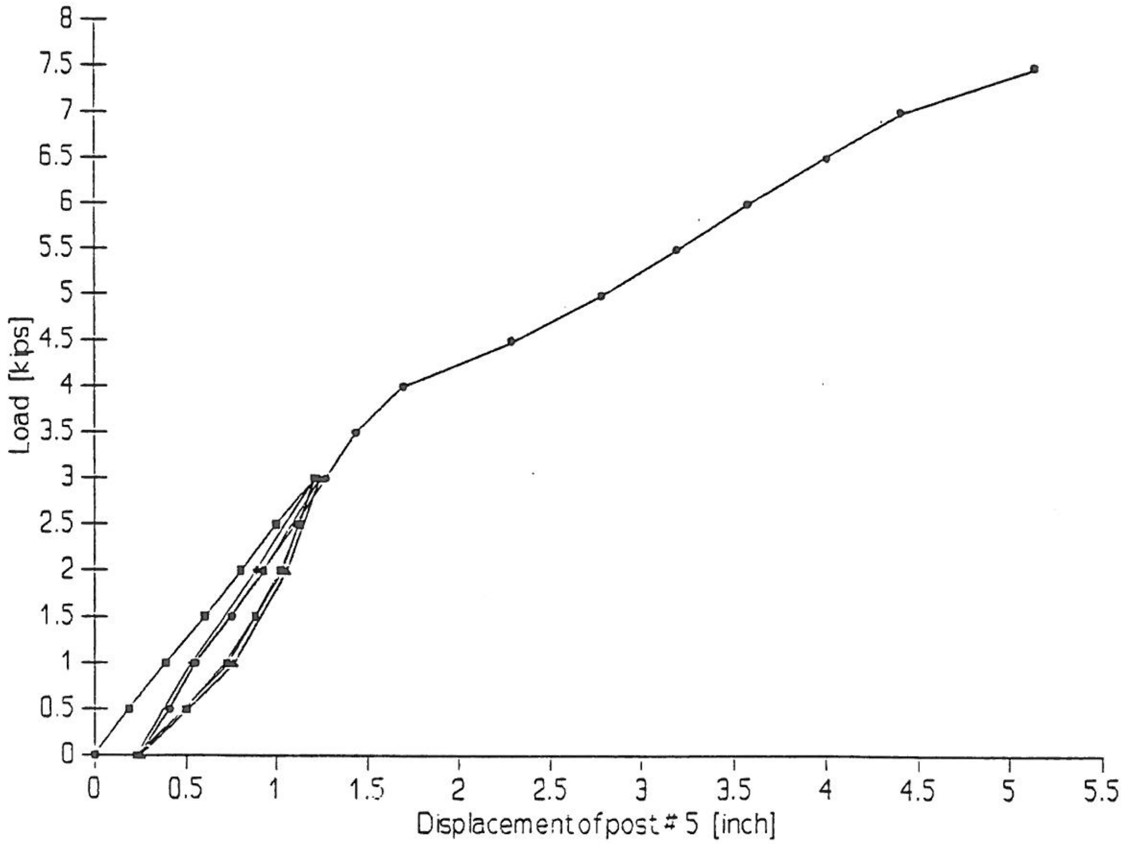

Figure 3.8. Hystereses Superposition of POST #5

The apparent action of the nut pulling through the post occurred between 3,500 and 4,000 pounds for POST #3. It occurred between 4,000 and 4,500 pounds for POST #5. The difference in the magnitude of the load range is attributed to the natural difference in properties of the two timber posts.

The detection of the rotation of the bridge deck from its supports occurred between 7,000 and 7,500 pounds on the graph. This load level is approximately 1,000 pounds higher than for the test of POST #3. The explanation for the difference is in the fact that in TEST #1, the rotation movement had been observed at the abutment near POST #1, completely at the opposite end of POST #5, location of this second test. Apparently the spike connection to the pile-cap at one end of the bridge deck was somewhat stronger than for the pile-cap at the other end.

Another comparison of these two tests can be made using the slopes of each specific section of the load-displacement graph.

Slopes of the **P**–Δ graph. [kips/in]		
	post # 3	post # 5
before nut penetration:	2.6	2.4
between nut penetration and deck rotation:	1.7	1.1
after beginning of deck rotation:	—	0.7

Slopes of the load-deflection graph for POST #5 are smaller than recorded for POST #3. This suggests either a slightly stiffer post or a stiffer post/curb/deck connection, or both, existed for POST #3 compared to POST #5. A stiffer post/curb/deck connection is a likely factor because of the position of POST #5 being located at the end of the specimen. For POST #5, the redistribution of the load could occur only on one side of the loaded post, i.e. towards POST #4. In contrast, POST #3 was centered in the middle of the structure length, where both adjacent posts could take part in the resisting effort.

Recommendations

With the close similarity of results observed from both tests, a step toward the improvement of the structure strength was implemented. Specifically, a square steel washer (4 in. x 4 in. x 0.25 in.) was placed between the post and the nut of the post/curb connection to prevent any bearing stress damage to the post.

Also, a reinforcement of the connection between the bridge deck and the pile-caps was introduced with one extra 1-inch diameter lag screw, 18-inch long, screwed in the middle of the deck cross-section at each support.

TEST #3 Performed on POST #3

Objective

By reloading POST #3, the middle post, it was possible to study the effect of some improvements introduced after the first two tests.

For this test, the supplementary washer was installed and the post/curb connection bolted back together. The two extra lag screws were also added to tighten the bridge deck to the pile-caps.

Displacement Measurement Instrumentation

The LVIT was replaced by a potentiometer. It is an electrical instrument with a variable resistance actuated by the advance of an inextensible wire attached to the system to be analyzed. By pulling on the wire, it is unwrapped from the axle of the instrument and the electrical resistance is modified proportionally to the distance pulled out.

Several reasons motivated the switch from an LVIT to a potentiometer. One reason was that five of these devices were in stock, ready to be used. Also, it would become important to collect data of the five posts simultaneously when testing the guardrail system with the rail attached to the posts. A potentiometer is also not subjected to any damage when the test specimen breaks suddenly. This is because the potentiometer was not positioned between the post and the winch but behind the bridge specimen (Figure 3.9). It is easier to use a potentiometer for measuring large magnitudes of displacement because, unlike an LVIT, it does not require a repositioning of the instrumentation. An LVIT is limited to three inches of travel, and must be repositioned when displacement causes that travel to be used up.

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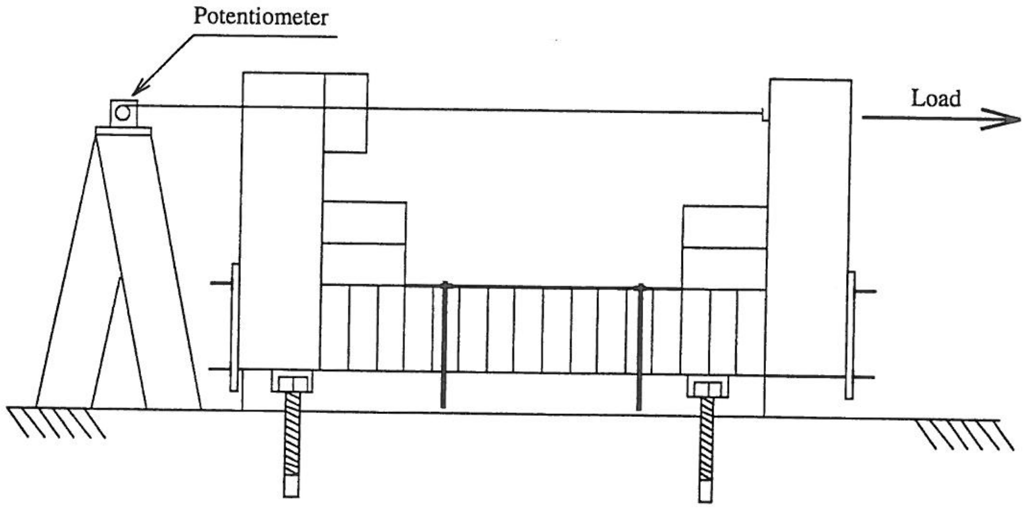


Figure 3.9. Location of the Potentiometer

A potentiometer also has some disadvantages. The accuracy is not as close as that of an LVIT (see Appendix A). Also, a potentiometer cannot collect data when the structure is moving towards the instrument (during unloading). This is because the potentiometer line is taut only when displacement is in a direction away from it (during loading). This was the reason for using an LVIT when studying the hystereses in the first two tests.

Results

The graph in Figure 3.10 shows the responses of POST #3 during TESTS #1 and #3, when loaded after 3 cycles of loading/unloading up to 3,000 pounds. Several observations can be made about the comparison of the slopes of the two load-displacement plots.

Before 3,500 pounds, the slope for both tests are quite similar, except for the shift between 1,500 and 2,000 pounds. At 3,500 pounds, the displacement is 22 percent greater in the TEST #3 on this post relative to the first test (δ_{T1} = 1.35 inches, δ_{T3} = 1.65 inches). This increase in displacement is probably due to an overall loosing of the post/curb/deck connection and can be compared with the hystereses phenomenon.

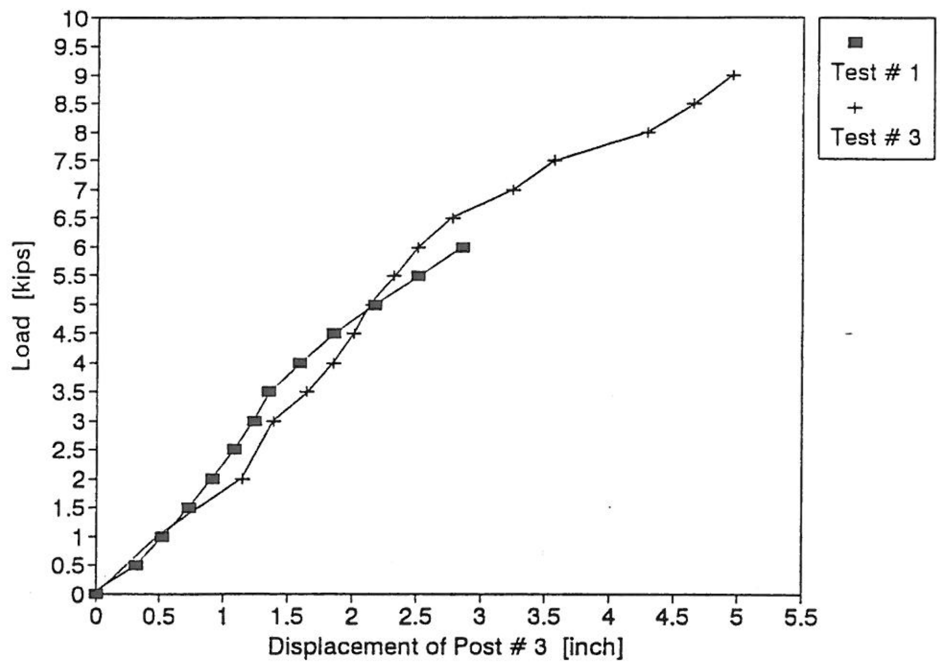


Figure 3.10. Comparison TEST #1 - TEST #3

After 3,500 pounds, the slope of the load-displacement plot for TEST #3 becomes steeper than for TEST #1. The effect of the washer is evident by preventing the nut of the post/curb connection from penetrating the post. Thus, the overall connection of the system is stiffer and, above 5,000 pounds of loading, the displacement of the post with the washer is smaller for TEST #1 compared to TEST #3.

After 6,500 pounds, the slope of the load-displacement plot for TEST #3 becomes noticeably flatter. At this load magnitude, because of the positioning of the potentiometer the uplift motion of the complete guardrail/bridge specimen increases the displacement values recorded for the post. Specifically, setting the potentiometer on the laboratory floor resulted in both the displacement of the post from the bridge deck and the displacement of the deck from the ground being recorded. The extra lag screw at each end of the deck only slightly improved the critical situation described at the end of TEST #1 (see "Results" - Page 18).

As expected, with the slightly less accuracy of a potentiometer instead of an LVIT, the slope of the load-displacement plot for TEST #3 is not as smooth as for TEST #1. This loss in accuracy is noticeable but not critical because the different slope discontinuities can still be observed.

Observations

The preceding tests constitute the conclusion of the series of tests performed on individual posts. The improvement of the post behavior caused by adding a washer was evident. Consequently washers were also used in the testing of the guardrail with the rail attached to the posts. Also, while turning around the guardrail specimen to test the system with the rail attached to it, a new tie-down of the deck to the floor was built to allow higher load magnitudes without causing the uplift motion of the specimen.

CHAPTER 4

TESTING OF THE COMPLETE POST/RAIL SYSTEM

In this phase of testing, the specimen was revolved 180 degrees in plan. This was done to allow testing of the guardrail on opposite side of the specimen. In contrast to the first side, the posts had not been previously loaded. A timber rail (6 in. x 11 in., actual size) was attached to the posts to complete the guardrail system. The middle post was used as the load point. Due to the presence of the rail member, the load was distributed to the deck through all posts. Thus, due to load sharing, an overall higher load capacity and a different mode of failure of the system were expected.

Connection of the Guardrail Specimen to the Floor

The pile-caps used as abutments in the preceding tests were retained. They were placed 24 feet apart under the bridge deck to allow space for a new system for connecting the specimen to the floor.

Figure 4.1 shows the improved connection of the deck to the floor. The system used the existing tie-down holes of the laboratory concrete slab. Each hole is drilled five feet from its neighbors on a grid pattern (see Figure 2.4). As the bridge specimen was 26 feet long, the central laminate of the deck was cut at each end to allow a 36-in.-long bolt, one per end of the deck, to be connected into the existing two-inch diameter tie-down holes. Two steel channels C12x30 (30-inch long), one at each end of the specimen, distributed the tightening force over the width of the deck.

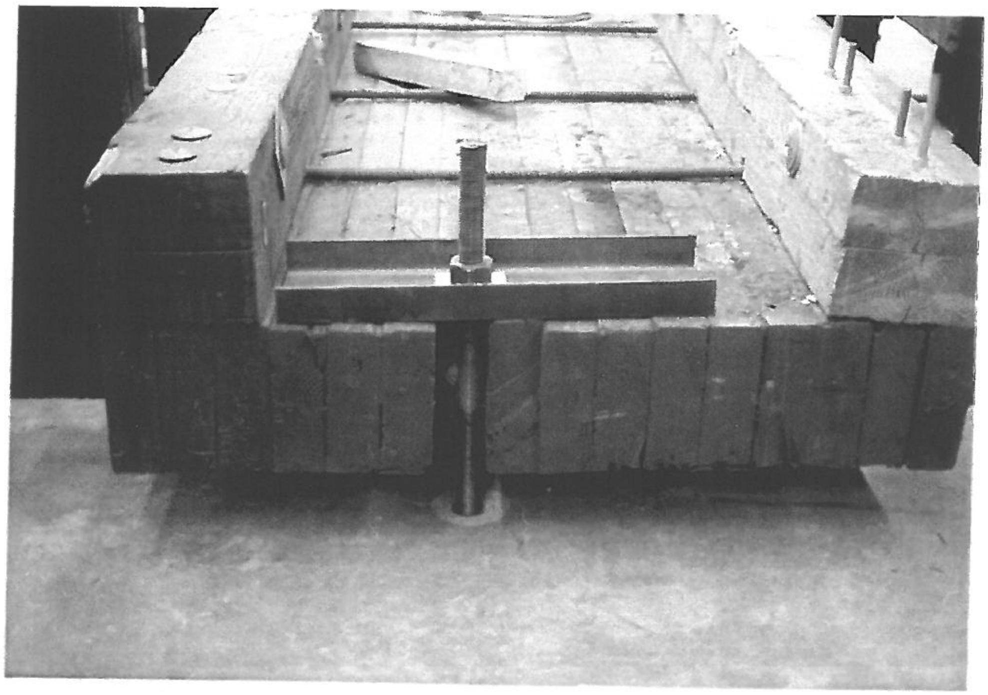


Figure 4.1. Picture of the New Connection of the Specimen to the Floor

Connection of the Loading Device to the Rail

Figure 4.2 shows the connection mechanism used to load the rail. The winch cable system described in Figure 2.6 was attached to the rail with a steel plate/cable system surrounding the rail/post connection. A steel plate, reinforced by a diagonal cross-shape bracing system, was set up on each side of the rail/post connection. The plates were linked together by 10 kips load capacity cables connecting opposite corners. By pulling

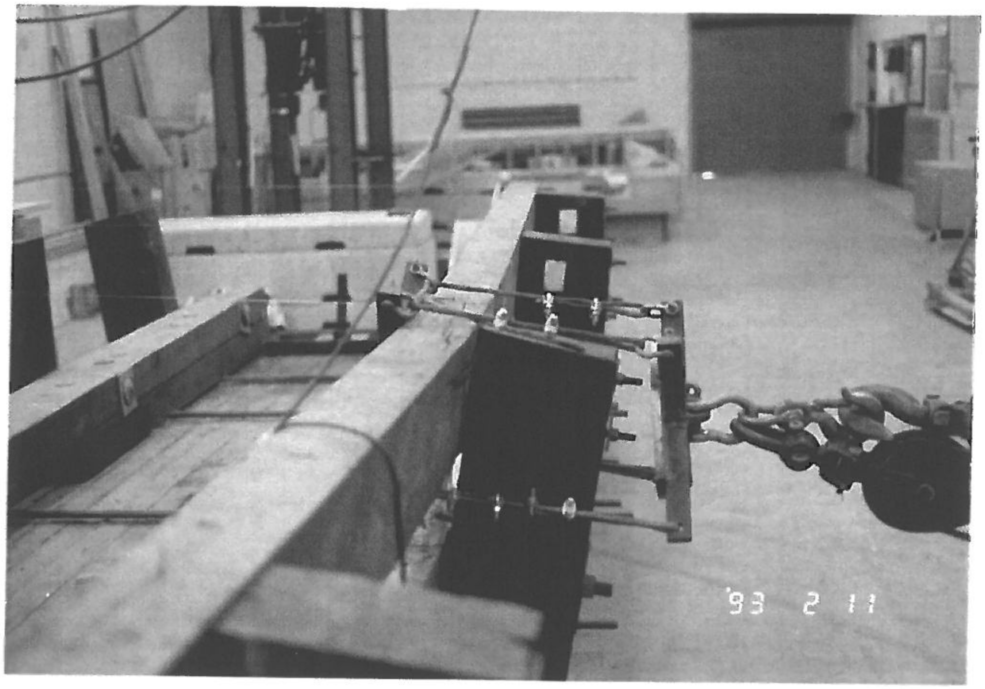


Figure 4.2. Picture of the Loading Device Around the Rail

outward at the center of the outside plate, the load was transferred to the inside plate and, thus, to the rail through bearing pressure. The plate against the rail distributed the load over a surface only slightly larger than the contact area of the rail/post connection. It can still be considered a concentrated load, compared to the spacing of the posts, for example. This plate also prevented any punching damage to the rail, which could have weakened the whole guardrail specimen.

Results

Deflection of the Posts

The graph in Figure 4.3 shows the response of POST #3 at each step of the loading procedure. Two distinct slope changes are observed. The first one occurs around 8,000 pounds and the second one at 24,000 pounds. One can also notice an irregular slope between 0 and 8,000 pounds. The explanations to those observations are:

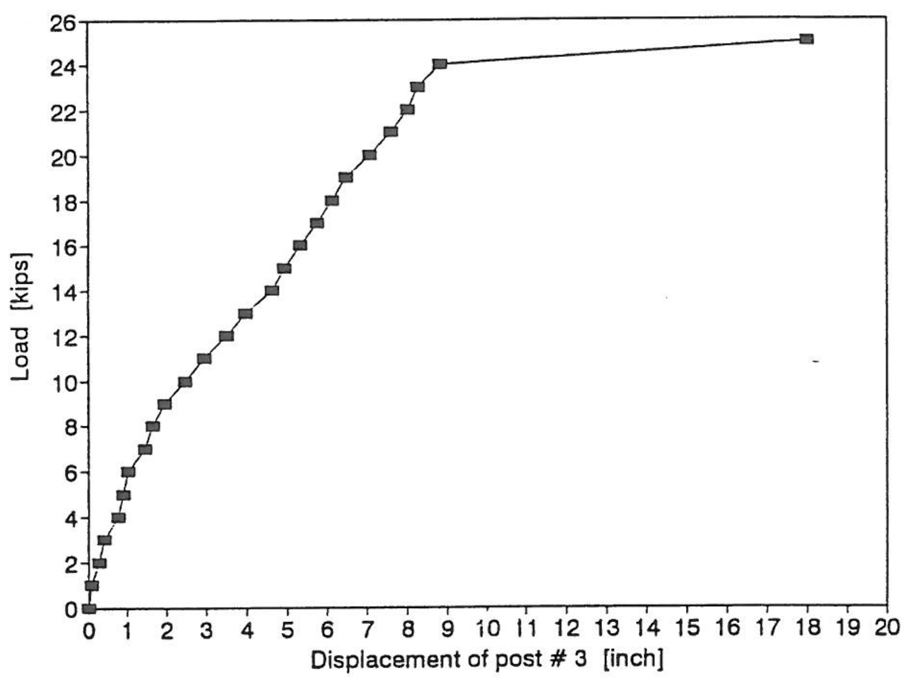


Figure 4.3. Deflection of Post #3

Irregularity of the slope before 8,000 pounds. When the load magnitude was relatively low (less than 8,000 pounds), the bolt at each end of the deck was tightened with enough strength to provide friction and resistance between the plate and the deck, between the deck and the pile-cap and between the pile-cap and the floor. As the load increased, either the deck moved relative to the cap or the entire specimen moved relative to the floor. Which motion took place depends on which interface frictional forces were overcome. Evidently, one of those events took place at 8,000 pounds. Thereafter, practically all the load was transferred to the ground through the bolt, which began to exhibit ductility as the load increased.

Discontinuity of the slope at 8,000 pounds. The reason for this phenomenon has been partially discussed above. The bending of the bolts connecting the specimen to the floor was responsible for the further increase in the magnitude of displacement of the post at each load increment. The reason of this slope discontinuity is not to be found in the behavior of the guardrail/bridge system itself. It is due to the measurement devices being set up on the laboratory floor instead of being installed directly on the bridge deck. Thus, the recorded displacements included the motion of the deck or specimen due to slippage, which is negligible or nonexistent in a real bridge with good connection of its deck to the abutments.

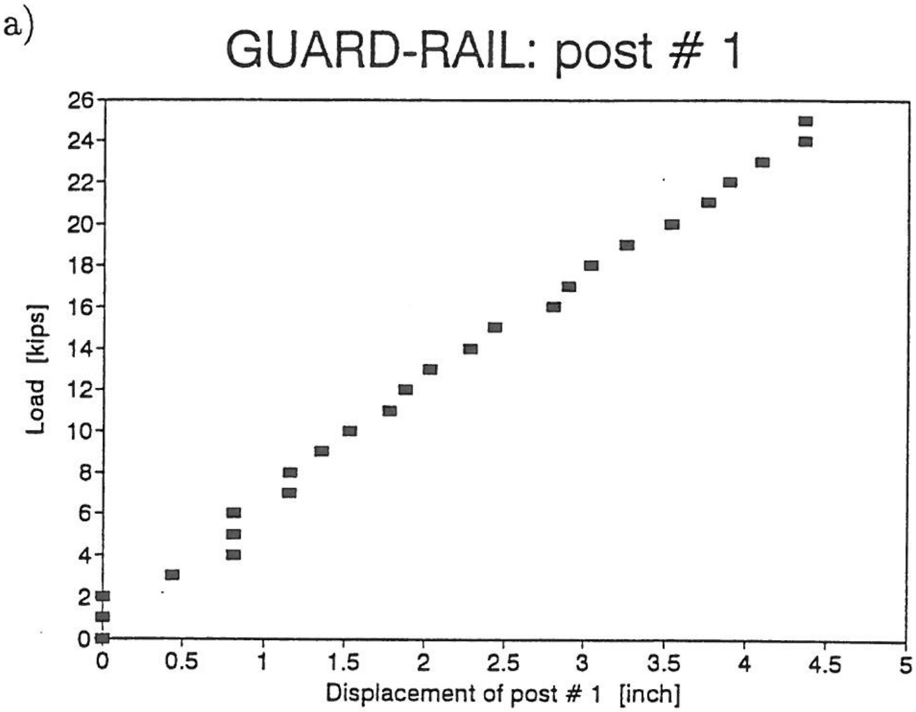
Discontinuity of the slope at 24,000 pounds. At this stage of the loading process, the rail simultaneously failed at the locations of POST #2 and POST #4. Afterwards, the sharing capacity of the system was considerably reduced and practically all the load was concentrated on POST #3 with the consequence of a rapid plastification of the post/curb connection.

Figure 4.4a and b, shows the measured responses of POST #1 and POST #5 during the same loading process up to failure of the guardrail system. The measured load displacement results for the two posts are compared before failure of the rail. The difference is 13 percent (δ_{T1} = 4.36 in., δ_{T5} = 3.86 in.) and this is attributed to the variability of material properties of the timber used and the fastening of the post/curb/deck connectors. The displacements of POST #2 and POST #4 (see Appendix B) are δ_{T2} = 6.02 in. and δ_{T4} = 5.89 in., respectively, a difference of two percent.

Displacement of the System

Figure 4.5 shows the positions of the five posts for each load increment of 4,000 pounds, starting at 8,000 up to 24,000 pounds. The last load magnitude shown in the graph, 25,000 pounds, occurred once the rail failed. The bending of the bolts connecting the bridge specimen to the floor participated for about two inches of the final displacement of the guard-rail system. The reading was obtained at the end of the testing, once the rail failed, by measuring the displacement of the deck due to the bending of the two bolts, starting around 8,000 pounds. Hence, all the deflection values recorded between 8,000 pounds and 25,000 pounds must be linearly reduced by an amount varying between 0 to 2 inches.

a)



b)

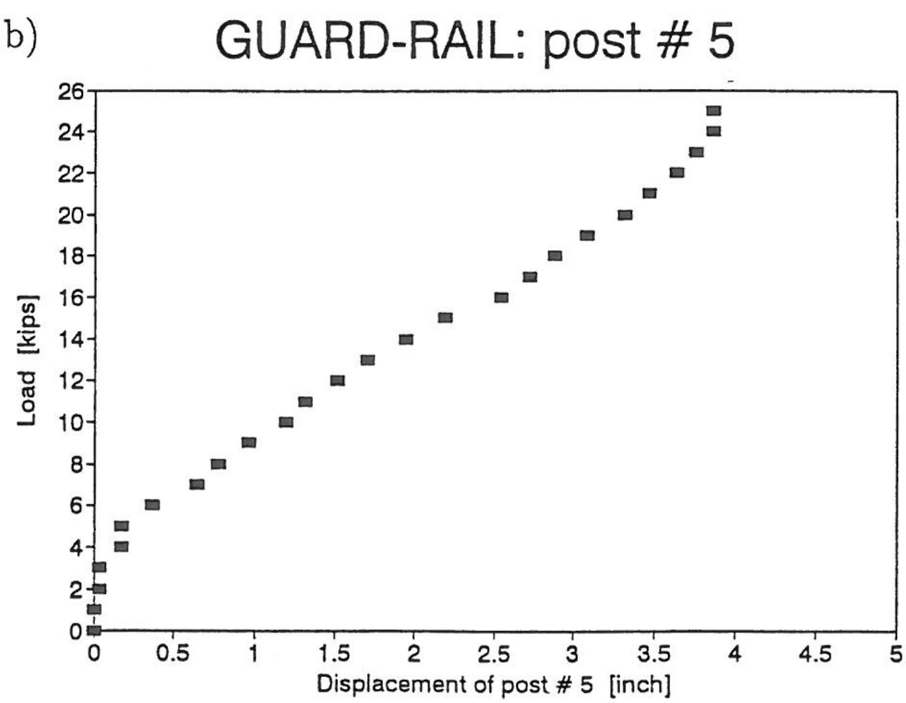


Figure 4.4: Deflection of posts # 1 and # 5

Using the values given by the graph in Figure 4.5 and the observation made about the bending of the bolts connecting the bridge specimen to the floor, the displacements of the posts at 24,000 pounds may be estimated as:

Deflection: ± 0.1 in.

Post #1	2.4 in.
Post #2	4.0 in.
Post #3	6.9 in.
Post #4	3.9 in.
Post #5	1.9 in.

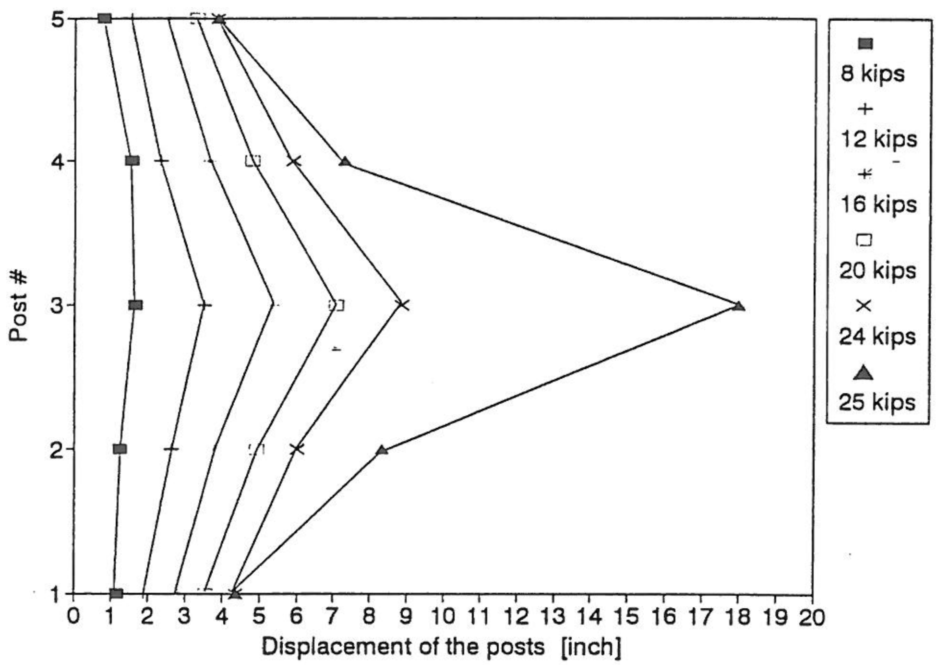


Figure 4.5. Deflection of the System

It is not appropriate to give more precision to the above results because the experimental determination of the displacement of the deck was made with a ruler with an accuracy of one sixteenth of an inch.

Mode of Failure of the System

Though the system was "noisy" from the beginning of the static loading, the first visible cracks in the rail appeared at about 20,000 pounds. At this load magnitude, gaps were noticed between the scuppers and the deck around POST #3 (Figure 4.6). The displacement of the loaded post created another visible gap between the core of the deck and the two laminates where the post was attached to by the spikes connection.

Those gaps are not critical for the bearing capacity of the guard-rail system and they can be closed by retightening the prestressing rods of the deck and the three-quarter-inch diameter bolts connecting the curb/scupper elements to the deck.

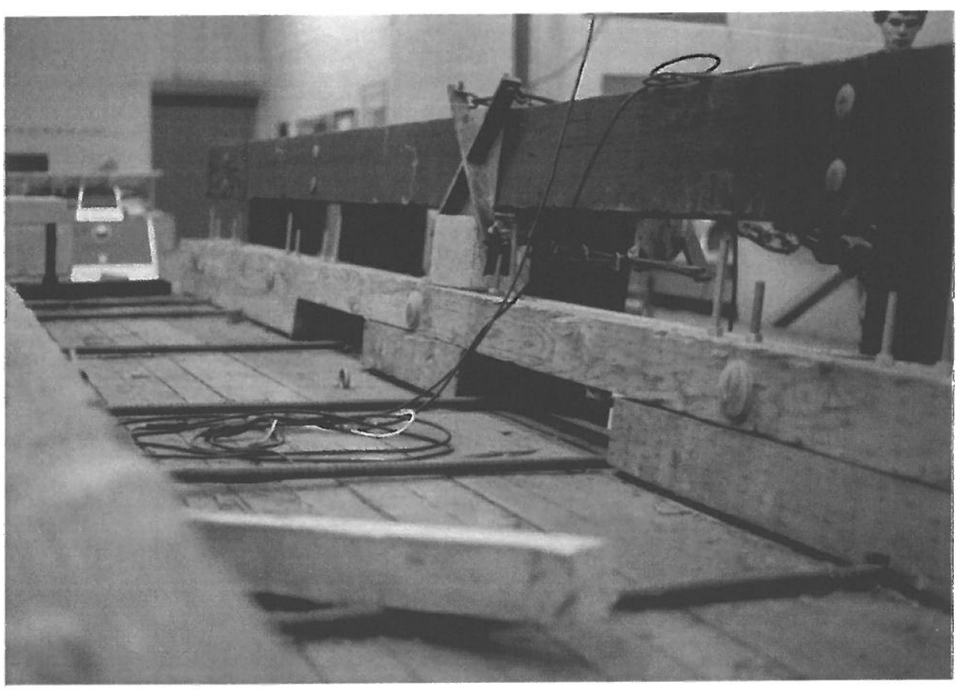


Figure 4.6. Gaps Around POST #3

The primary failure of the system occurred by rupture of the 6 in. x 11 in. timber rail at the locations of POSTS #2 and #4 (Figure 4.7). This failure then caused all the load to be concentrated on POST #3. This sudden redistribution of load to POST #3 induced a bearing failure in the curb/scupper elements. This failure occurred between the 24,000 and 25,000 pounds load levels.

Just after the rail began to split apart at the location of POSTS #2 and #4, the two posts adjacent to the loaded post split in half due to the motion of the rail (Figure 4.7). The small in-plane rotation of the rail induced perpendicular-to-grain tension in each of the adjacent posts because they were prevented from rotating by their tightening connections to the deck and curbs.

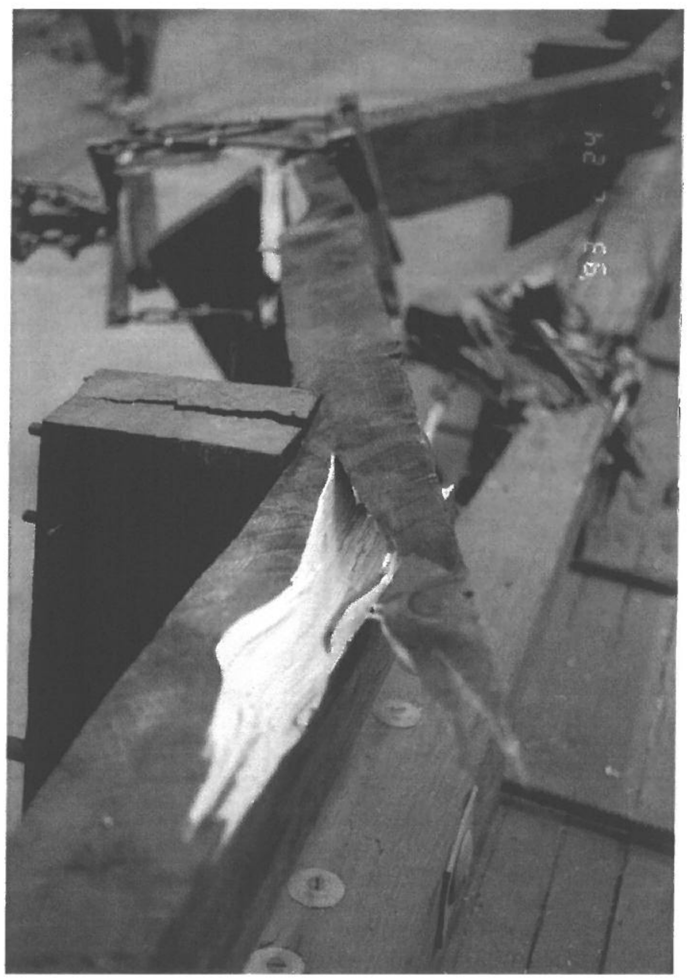


Figure 4.7 Failure of the Rail

CHAPTER 5

CONCLUSIONS

The mode of failure of a single post and its load capacity were determined during a set of tests. To improve the bolted connection of the post to the deck, a square washer (16 in^{2}) was placed under the bolt connecting the post to the curb of the bridge specimen. With this improvement, the mode of failure of individual posts could not be determined due to the vertical motion of the entire guard-rail specimen out of its abutments.

Several difficulties occurred with the connection of the bridge specimen to the floor. A satisfactory solution, using the tie-downs of the laboratory concrete slab, was determined for conduct of the second part of the experimentation. This enabled the determination of the ultimate capacity of the complete guard-rail system and its mode of failure.

The guard-rail system investigated in the load tests permits large rotations of the posts under static load. This is due to the short resisting moment arm of the post/deck connection and to the rotational flexibility of the deck members. The rotation of the longitudinal deck members was reduced by transversely post-tensioning the deck.

The load capacity of the system was reached between 24,000 and 25,000 pounds. The AASHTO perpendicular to rail load requirement being 10,0000 pounds, the inferred safety factor for the specimen is between 2.4 and 2.5. This factor of safety is not unusual and is considered normal, or possibly low, for wood systems. The initial rupture of the system occurred when the rail split at locations immediately next to the two posts that were adjacent to the loaded post. One side-effect of the relatively large deformation capacity of the system is that, at time of failure, the four adjacent posts were split along their length because of the occurrence of perpendicular-to-grain tension induced by the in-plane rotation of the rail.

H:6

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APPENDIX A

Individual Posts Data

The calibration of the LVIT (first two tests) and the potentiometer (last test) gave the following results for the three posts tested individually:

LVIT: 0.061 Volt = 1/20 inch ±0.01 inch

Potentiometer: 0.084 Volt = 1 inch ±0.05 inch

	Post # 3		
Load [kips]	Unloaded [volt]	Loaded [volt]	Displ. [inch]
0	2.949	2.949	0
0.5	2.949	3.334	0.32
1	2.949	3.588	0.52
1.5	2.949	3.839	0.73
2	2.949	4.055	0.91
2.5	2.949	4.267	1.08
3	2.949	4.459	1.24
2	2.949	4.241	1.06
1	2.949	3.877	0.76
0	2.949	3.256	0.25
0.5	2.949	3.456	0.42
1	2.949	3.684	0.60
1.5	2.949	3.893	0.77
2	2.949	4.085	0.93
2.5	2.949	4.274	1.09
3	2.949	4.456	1.24
2	2.949	4.246	1.06
1	2.949	3.885	0.77
0	2.949	3.281	0.27
0.5	2.949	3.474	0.43
1	2.949	3.697	0.61
1.5	2.949	3.909	0.79
2	2.949	4.100	0.94
3	2.949	4.452	1.23
2	2.949	4.242	1.06
1	2.949	3.880	0.76
0	2.949	3.291	0.28
0.5	2.949	3.484	0.44
1	2.949	3.711	0.62
1.5	2.949	3.920	0.80
2	2.949	4.098	0.94
3	2.949	4.463	1.24
2	2.949	4.237	1.06
1	2.949	3.885	0.77
0	2.949	3.285	0.28
1	2.949	3.713	0.63
2	2.949	4.109	0.95
3	2.949	4.465	1.24
3.5	4.851	4.723	1.35
4	4.851	4.432	1.59
4.5	4.851	4.113	1.85
5	4.851	3.722	2.17
5.5	4.851	3.300	2.51
6	4.851	2.888	2.85

Load:
The load is applied perpendicular to the bridge driveway at post # 3, 25 inches above the deck.

Unloaded:
Voltage of the LVIT before applying the load.

Loaded:
Voltage of the LVIT when the load is applied.

Displ:
Horizontal displacement of the post at the location of the load.

table saved in A:POST3.1

<-- repositioning of the LVIT.

Table A.1. Load-Displacement of POST #3, TEST #1

Load [kips]	Post # 5 Unloaded [volt]	Loaded [volt]	Displ. [inch]
0	3.135	3.135	0
0.5	3.135	3.372	0.19
1	3.135	3.606	0.39
1.5	3.135	3.865	0.60
2	3.135	4.106	0.80
2.5	3.135	4.360	1.00
3	3.135	4.608	1.21
2.5	3.135	4.514	1.13
2	3.135	4.376	1.02
1.5	3.135	4.210	0.88
1	3.135	4.013	0.72
0.5	3.135	3.745	0.50
0	3.135	3.413	0.23
1	3.135	3.787	0.53
2	3.135	4.217	0.89
3	3.135	4.629	1.22
2	3.135	4.407	1.04
1	3.135	4.028	0.73
0	3.135	3.434	0.25
1	3.135	3.789	0.54
2	3.135	4.275	0.93
3	3.135	4.649	1.24
2	3.135	4.431	1.06
1	3.135	4.061	0.76
0	3.135	3.452	0.26
0	3.135	3.443	0.25
0.5	3.135	3.638	0.41
1	3.135	3.808	0.55
1.5	3.135	4.047	0.75
2	3.135	4.264	0.93
2.5	3.135	4.486	1.11
3	3.135	4.683	1.27
3.5	4.882	4.683	1.43
4	4.882	4.369	1.69
4.5	4.882	3.647	2.28
5	4.882	3.044	2.78
5.5	4.590	4.094	3.18
6	4.590	3.630	3.56
6.5	4.590	3.104	3.99
7	4.698	4.205	4.40
7.5	4.698	3.312	5.13

Load:
The load is applied perpendicular to the bridge driveway at post # 5, 25 inches above the deck.

Unloaded:
Voltage of the LVIT before applying the load.

Loaded:
Voltage of the LVIT when the load is applied.

Displ:
Horizontal displacement of the post at the location of the load.

table saved in A:POST5.1

<-- reading after a 15 minute interruption. (at second 0 row)

<-- repositioning of the LVIT. (after 3 kips)

<-- repositioning of the LVIT. (after 5 kips)

<-- repositioning of the LVIT. (after 6.5 kips)

Table A.2. Load-Displacement of POST #5, TEST #2

	Post # 3		
Load [kips]	Unloaded [volt]	Loaded [volt]	Displ. [inch]
0	4.081	4.081	0
1	4.081	4.124	0.51
2	4.081	4.178	1.15
3	4.081	4.198	1.39
3.5	4.081	4.220	1.65
4	4.081	4.236	1.85
4.5	4.081	4.250	2.01
5	4.081	4.262	2.15
5.5	4.081	4.276	2.32
6	4.081	4.292	2.51
6.5	4.081	4.314	2.77
7	4.081	4.353	3.24
7.5	4.081	4.380	3.56
8	4.081	4.441	4.29
8.5	4.081	4.472	4.65
9	4.081	4.498	4.96

Load:
The load is applied perpendicular to the bridge driveway at post # 3, 25 inches above the deck.

Unloaded:
Voltage of the potentiometer gage before applying the load.

Loaded:
Voltage of the potentiometer gage when the load is applied.

Displ:
Horizontal displacement of the post at the location of the load.

table saved in A:POST3.2

Table A.3. Load-Displacement of POST #3, TEST #3

Appendix B

Guard-rail System Data

The calibration of the potentiometers used in the second part of the experiment gave the following:

- **Post # 1:** 1 Volt = 6.85″ ±0.16″
- **Post # 2:** 1 Volt = 4.75″ ±0.06″
- **Post # 3:** 1 Volt = 11.47″ ±0.05″
- **Post # 4:** 1 Volt = 11.09″ ±0.30″
- **Post # 5:** 1 Volt = 6.86″ ±0.04″

Load: [kips]	Post # 1 [volt]	[inch]	Post # 2 [volt]	[inch]	Post # 3 [volt]	[inch]	Post # 4 [volt]	[inch]	Post # 5 [volt]	[inch]
0	5.716	0	9.790	0	2.440	0	5.436	0	10.137	0
1	5.716	0	9.790	0	2.449	0.10	5.436	0	10.137	0
2	5.716	0	9.764	0.12	2.464	0.28	5.436	0	10.132	0.03
3	5.652	0.44	9.740	0.24	2.476	0.41	5.436	0	10.132	0.03
4	5.598	0.81	9.694	0.46	2.507	0.77	5.506	0.78	10.112	0.17
5	5.598	0.81	9.675	0.55	2.517	0.88	5.506	0.78	10.112	0.17
6	5.598	0.81	9.629	0.76	2.528	1.01	5.506	0.78	10.084	0.36
7	5.547	1.16	9.575	1.02	2.565	1.43	5.574	1.53	10.043	0.64
8	5.546	1.16	9.532	1.23	2.583	1.64	5.574	1.53	10.024	0.78
9	5.518	1.36	9.449	1.62	2.610	1.95	5.574	1.53	9.997	0.96
10	5.492	1.53	9.371	1.99	2.655	2.47	5.574	1.53	9.963	1.19
11	5.456	1.78	9.296	2.35	2.697	2.95	5.646	2.33	9.946	1.31
12	5.441	1.88	9.236	2.63	2.745	3.50	5.646	2.33	9.916	1.52
13	5.419	2.03	9.188	2.86	2.785	3.96	5.646	2.33	9.889	1.70
14	5.383	2.28	9.112	3.22	2.843	4.62	5.646	2.33	9.853	1.95
15	5.361	2.43	9.079	3.38	2.870	4.93	5.763	3.63	9.818	2.19
16	5.307	2.80	8.994	3.78	2.905	5.33	5.763	3.63	9.767	2.54
17	5.293	2.90	8.926	4.10	2.940	5.74	5.763	3.63	9.740	2.72
18	5.273	3.03	8.865	4.39	2.975	6.14	5.763	3.63	9.717	2.88
19	5.241	3.25	8.815	4.63	3.006	6.49	5.763	3.63	9.688	3.08
20	5.201	3.53	8.754	4.92	3.056	7.07	5.868	4.79	9.654	3.31
21	5.168	3.75	8.695	5.20	3.103	7.60	5.868	4.79	9.632	3.46
22	5.148	3.89	8.648	5.42	3.141	8.04	5.868	4.79	9.608	3.63
23	5.119	4.09	8.608	5.61	3.163	8.29	5.967	5.89	9.590	3.75
24	5.080	4.36	8.522	6.02	3.211	8.84	5.967	5.89	9.575	3.86
25	5.080	4.36	8.040	8.31	4.008	17.98	6.092	7.28	9.575	3.86

Load:	The load is applied perpendicular to the bridge driveway at the connection of the post # 3 and the guard-rail.
[volt]	Voltage of the potentiometer gages during the loading process.
[inch]	Horizontal displacement at the connection of the guard-rail with each post.

table saved in A:GUARD.POS

Table B.1. Load-Displacement of the Five Posts