SEISMIC RETROFIT AND REHABILITATION OF THE MILLION DOLLAR BRIDGE

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Abstract

The Million Dollar Bridge carries the Copper River Highway across the Copper River, near Cordova, Alaska. The bridge was built in 1909-1910 to carry the Copper River and Northwestern Railway. It was badly damaged in the Prince William Sound earthquake of March 27, 1964. It was listed on the National Register of Historic Places on March 31, 2000. This paper describes the rehabilitation and seismic retrofit of the bridge. This includes raising the fallen Span 4, replacement of the damaged Pier 3 with a new pier founded on a new pile foundation, and seismic isolation of the bridge with friction pendulum isolation bearings.

Introduction

The Million Dollar Bridge carries the Copper River Highway across the Copper River, near Cordova, Alaska. The bridge is shown in Figure 1. It consists of four Pratt truss spans measuring 400', 300', 450', and 400' (122 m, 91 m, 137 m, and 122 m) between centerlines of pins, from south to north of the bridge. The arrangement of spans is shown in Figure 2. The spans are numbered 1-4 from south to north (left to right in the figure). The bridge is supported in the river on three massive concrete piers, numbered 1-3 from south to north.

The bridge was built in 1909-1910 by the Katalla Corporation to carry the Copper River and Northwestern Railway from Cordova to the Kennecott/Bonanza copper mine. The bridge was first called the Miles Glacier Bridge; it takes its current name from the cost of construction of the bridge, which was actually $1,424,774 (Quinn).

The bridge is particularly noteworthy for the climatic conditions under which in was built; much of the construction was done in the winter of 1909-1910. The caisson for construction of Pier 1 was lowered to the bottom of the river on May 4, 1909. Span 4 was completed on June 19, 1910, thirteen months later. Between September 26 and April 1 the wind blew at a velocity of 60 to 96 miles per hour (96 to 154 kph), *most of the time* (Engineering Record, Dec. 1910) (emphasis added). Wind chill temperatures frequently approached -60°F (-51°C) (Quinn). The original construction of the bridge is well documented in Engineering Record articles published contemporaneously with the construction (Engineering Record, Aug. 1910).

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The Copper River and Northwestern Railway ceased operations in 1938 when the Kennecott mines were closed. The small population along the railway alignment did not make it financially feasible to operate the railway for passengers and freight. The bridge was converted to a road bridge in 1958 by the (then) Bureau of Public Roads.

The bridge was badly damaged in the Prince William Sound earthquake (Mw 9.2) on Good Friday March 27, 1964. The southern end of Span 4 fell off of Pier 3 into the Copper River and Span 3 was shifted several feet on top of the pier. Pier 3 was tilted several degrees from the vertical and the top part of the pier was sheared several feet relative to the bottom part. All of the rocker bearings were toppled and all of the bolts at fixed bearings were sheared. Because of the severe damage the bridge was closed to traffic. The volume of traffic using the bridge did not justify repairing it at the time.

The bridge was listed on the National Register of Historic Places on March 31, 2000. This is the nations official list of cultural resources worthy of preservation. The listing has prompted the Alaska Department of Transportation & Public Facilities to rehabilitate the bridge and perform a seismic retrofit so that it won't be destroyed in a future earthquake. The rehabilitation and retrofit must be done in a way that respects the historic nature of the structure, generally in accordance with "The Secretary of the Interior's Standards for Rehabilitation," and the "Guidelines for Rehabilitating Historic Buildings" (Secretary of the Interior).

Rehabilitation and Seismic Retrofit Measures

The most obvious repair to the bridge is the raising of the fallen Span 4 (see Figure 1). As illustrated in Figure 3, the span was raised using temporary support towers built upstream and downstream of the fallen span and a lifting beam passed underneath the truss. The lifting beam engaged the upstream and downstream faces of the truss at a panel point; at pins connecting the bottom chords, verticals, and diagonals. The lifting beam was raised using a system of jacks and high-strength rods operating from the support towers. Thus, the span was raised to its original position.

The support towers were built in the fall of 2003 and spring of 2004 and the span lifted before the Copper River rose too high. During the summer and fall of 2004, the truss will be repaired while it's temporarily supported. The bottom chords of the truss were badly bent when the span fell, and many of the bottom lateral braces were smashed. These members (see Figure 4) will be replaced with new, built-up members similar to the original members. This is in order to respect the historical integrity of the structure. Also, several members at the end of Span 3 will be replaced; these were damaged when the span fell off of its bearings onto Pier 3.

The other main repair of the bridge will be the replacement of the damaged Pier 3 with a new pier. The main cause of the failure of the bridge in the 1964 earthquake may have been liquefaction of the river sands beneath Pier 3. This would explain the observed

5° tilt of the pier and the inferred tilt (from various evidence) of the caisson supporting it. It's possible that Span 4 fell off of the pier because of the tilt. In any case, the span hit the pier as it fell, breaking it along a construction joint (see Figure 1). The most cost effective and permanent repair is to remove the existing pier and replace it with a new hollow pier supported on a pile foundation, as illustrated in Figure 5. This new foundation will bridge over the old caisson, which will be abandoned. The new piles will be tipped 150' (46 m) below the riverbed, below the zone of liquefiable sands.

The seismic retrofit is intended to protect the bridge against ground motions with a return period of 475 years. This hazard corresponds to magnitude 8+ events on the 1964 segment (i.e., the segment that slipped in the 1964 earthquake) and the adjacent Yakataga segment of the Alaska-Aleutian megathrust subduction zone. One of the ground motions used for dynamic analysis of the bridge is shown in Figure 6. This is based on the Vina del Mar record of the 1985 Valparaiso (Chile) earthquake (M_s=7.8). The motion has been made compatible with a target spectrum corresponding to the 475-year hazard.

The main element of the seismic retrofit of the bridge will be the installation of friction pendulum isolation bearings (see Figure 7). All of the existing fixed and rocker bearings were destroyed during the 1964 earthquake and must be replaced. Although friction pendulum bearings are more expensive ($30,000 each) than other bridge bearings they're cost effective in this case because they'll minimize the number of piles required to support the new Pier 3. These 72" (1.83 m) diameter pipe piles will cost about $300,000 each because of the difficult site conditions.

Originally, each truss (or each face of each truss) was supported on an individual bearing at each pier. There isn't sufficient space on top of the piers to support each truss on an individual friction pendulum bearing, however. These will be approximately 90 inches (2.29 m) in diameter in order to accommodate earthquake displacement demands. Therefore, as illustrated in Figure 7, adjacent trusses will share a common bearing at each pier (transversely, there will be a bearing under each face of the trusses). The end pin of each truss will be supported in a new truss shoe that is supported in turn on top of the friction pendulum bearing. The dimensions of the shoe and the positions of the pin will be set to balance the dead loads of the spans about the centerline of the bearing. In order to accommodate unbalanced loads, live loads, seismic, and other loads; a structural link will be installed that connects the bottom chords of the trusses together, making them continuous across the bearing.

Figure 8 shows a schematic drawing of a friction pendulum bearing. These consist of a stainless steel concave dish and a stainless steel articulated slider surfaced with a composite liner. During an earthquake the slider moves back and forth on the concave dish; the spherical surfaces of the slider and the dish define a motion similar to that of a pendulum. The composite liner produces a frictional force that is 5-7% of the vertical force acting on the bearing. A friction pendulum bearing isolates a structure from an earthquake through pendulum motion and absorbs earthquake energy through friction.

Another element of the seismic retrofit of the bridge will be the strengthening of Piers 1 & 2 of the bridge. These are massive, unreinforced concrete structures similar to the damaged Pier 3; Pier 2 is visible in the foreground of Figure 1. As illustrated in Figure 9, Piers 1 & 2 will be strengthened with vertical high-strength rods. At each pier, bundles of three 1¼-inch (32 mm) rods will be grouted into 5½-inch (140 mm) diameter holes cored through the pier and extending into the caisson below. Thus, the flexural strength of the pier will be increased and rocking of the pier on the caisson prevented. Rocking of the piers would be undesirable in view of the friction pendulum bearing retrofit. When rocking is prevented, the bridge will have a single, clearly defined, inelastic mechanism—as recommended by ATC-32 (Applied Technology Council). Strengthening is also desirable because the piers contain cold joints at unknown locations (the near failure of Pier 3 along a cold joint may be seen in Figure 1).

Analysis for Seismic Retrofit

Because friction pendulum bearings are highly nonlinear devices the demands on the structure were computed using time history analysis (Bathe). The analysis was performed using the ADINA general-purpose finite element program (ADINA R&D). The ADINA model of the bridge is shown in Figure 10.

The truss models are detailed three-dimensional beam element models. All of the truss members, other than eye-bars, were modeled as beam elements with flexural and torsional stiffness. Major joints were modeled with member eccentricity and rigid end zones representing the connections of members with gusset plates. The concrete deck was modeled with shell elements connected to the stringer elements with rigid links.

A proprietary database product—created in Microsoft Access—was used to compute the cross-sectional properties of members and to generate the models of the trusses. Using the database, built-up cross-sections were defined by describing the individual plates, channels, angles, etc. from which the cross-section is made. A screen-shot from the database is shown in Figure 11. Cross-sectional properties are computed automatically, and if a cross-section is assigned to a member, the capacity of that member is computed automatically also.

The database was used to generate that part of the ADINA input file with the modeling of the trusses. The database was also used to scan the analysis result files and extract the peak forces (e.g., the peak tensile and compressive forces with the corresponding bending moments) for checking against capacity. The database computed demand-capacity ratios for all of the members and summarized them according to member type.

The concrete deck was added to the structure when it was converted from a railway to a road bridge. Because it was cast in a plastic state, the deck doesn't carry any of the dead load. The trusses carry all of it. This condition was replicated in the model of the bridge using the birth-and-death feature of ADINA. A "dummy" deck, modeled with a flexible material (but having the density of concrete), was birthed initially so that the trusses would carry the entire dead load. In a subsequently time step, the "real" deck, modeled using a material with the elasticity of concrete, was birthed, and the "dummy" deck simultaneously killed. This effectively replaced the flexible material with the concrete material. The deck was then composite with the truss in the time history analysis (a restart analysis from the dead load state).

The friction pendulum bearings were modeled using the contact surface model (Ingham) shown in Figure 12. In this model, the bearing dish is modeled with a spherical mesh of contact segments that together constitute a contact surface. The bearing slider is modeled with a single contact point that exists on a contact segment (surface) that lies on one face of a solid finite element. This solid element and its mirror image define the body of the slider. The opposing contact surfaces are defined as a contact pair with a coefficient of friction equal to that specified for the bearing. This modeling faithfully reproduces the force-deformation behavior of the bearing, including the dependence of the restoring force (arising from the curvature of the bearing) and the frictional force on the instantaneous vertical force acting on the bearing. The modeling also captures the bi-directional coupling of the frictional force (Chaudary).

The piles supporting the new Pier 3 are modeled with inelastic moment-curvature beam elements over their whole length. Those elements are supported by discrete springs representing the inelastic response of the surrounding soil, both laterally (p-y springs) and vertically (t-z springs). The properties of the springs are for liquefied conditions and unliquefied conditions in separate analyses. The database was also used to generate the input file for modeling the piles and soil springs.

Figure 13 shows the effect of seismic isolation on the forces in the superstructure. The abscissa in the plot is the ratio of un-isolated member demand to the member demand when the superstructure is isolated with friction pendulum bearings. The graph is a histogram showing the numbers of members with different demand ratios. The average demand ratio is 0.54. Essentially elastic response of the trusses is expected after isolation. Isolation of the spans is also effective in reducing the forces in the remaining Piers 1 & 2, but less so; the average demand ratio is 0.64. This is because the piers are massive concrete elements and a large part of the demand on them is from their own inertia. Because they are completely unreinforced they will have to be strengthened with high-strength rods in spite of the isolation of the superstructure. The isolation is very effective in minimizing the number of piles required to support the new Pier 3, however.

Figure 14 shows the hysteresis loops formed by plotting the transverse force in one of the friction pendulum bearings against the transverse movement of the same bearing. The loops produced by the contact surface model (see Figure 12) deviate significantly from the idealized hysteresis loop shown in Figure 8. This is because the contact surface model faithfully reproduces the dependence of the restoring and frictional forces on the vertical force acting on the bearing and the bi-directional coupling of the frictional force. The vertical force on the bearing varies considerably during an earthquake because of overturning of the structure and because of shaking of the structure induced by vertical ground motion. The idealized hysteresis loop—which is frequently assumed in analysis—implicitly ignores the dependence of the lateral forces on the vertical force and the coupling of the frictional force.

Summary

The Million Dollar Bridge was badly damaged in the Prince William Sound earthquake of March 27, 1964. It was listed on the National Register of Historic Places on March 31, 2000. The rehabilitation of the bridge, now underway, includes raising the fallen Span 4 and replacement of damaged members in that span. It includes replacement of the damaged Pier 3, and strengthening of Piers 1 & 2. The seismic retrofit of the bridge includes replacement of the original rocker bearings with friction pendulum bearings. A contact surface model of the bearings was used to accurately model their response to overturning of the structure and vertical ground motion. Time history analysis shows that the friction pendulum bearings are an effective seismic retrofit measure.

References

1. ADINA R&D Inc., ADINA Theory and Modeling Guide, February 1995.

2. Applied Technology Council, Improved Seismic Design Criteria for California Bridges: Provisional Recommendations, Report No. ATC-32, 1996.

3. Bathe, K.J., Finite Element Procedures, Prentice Hall, 1996.

4. Chaudhary, A.B., Bathe, K.J., A Solution Method for Static and Dynamic Analysis of 3- Dimensional Contact Problems With Friction, Computers & Structures 24: 855-873, 1986.

5. Ingham, Tim J., Modeling of Friction Pendulum Bearings for the Seismic Analysis of Bridges, First M.I.T. Conference on Computational Fluid and Solid Mechanics, Cambridge, MA, 2001.

6. Quinn, Alfred O., Iron Rails to Alaska Copper, D'Aloquin Publishing Co., NY.

7. Engineering Record, Incidents in the Construction of the Miles Glacier Bridge, Engineering Record, Vol. 62, No. 27, December 31, 1910.

8. Engineering Record, The Miles Glacier Bridge, Engineering Record, Vol. 62, No. 6, August 6, 1910.

9. Secretary of the Interior, Standards for Rehabilitation & Illustrated Guidelines for Rehabilitating Historic Buildings; http://www2.cr.nps.gov/tps/tax/rhb/index.htm.

Figure 1: The Million Dollar Bridge, Showing Piers 2 & 3 and the Fallen Span 4.

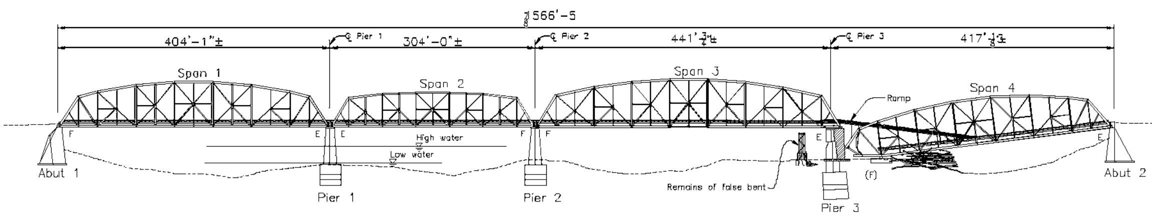

Figure 2: Span Arrangement.

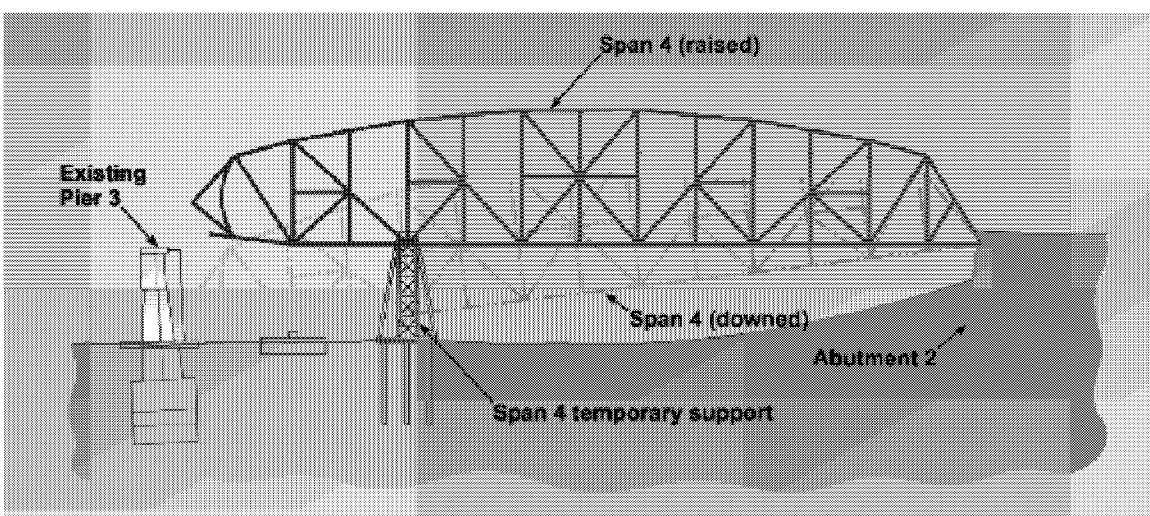

Figure 3, Lift and Temporary Support of Span 4.

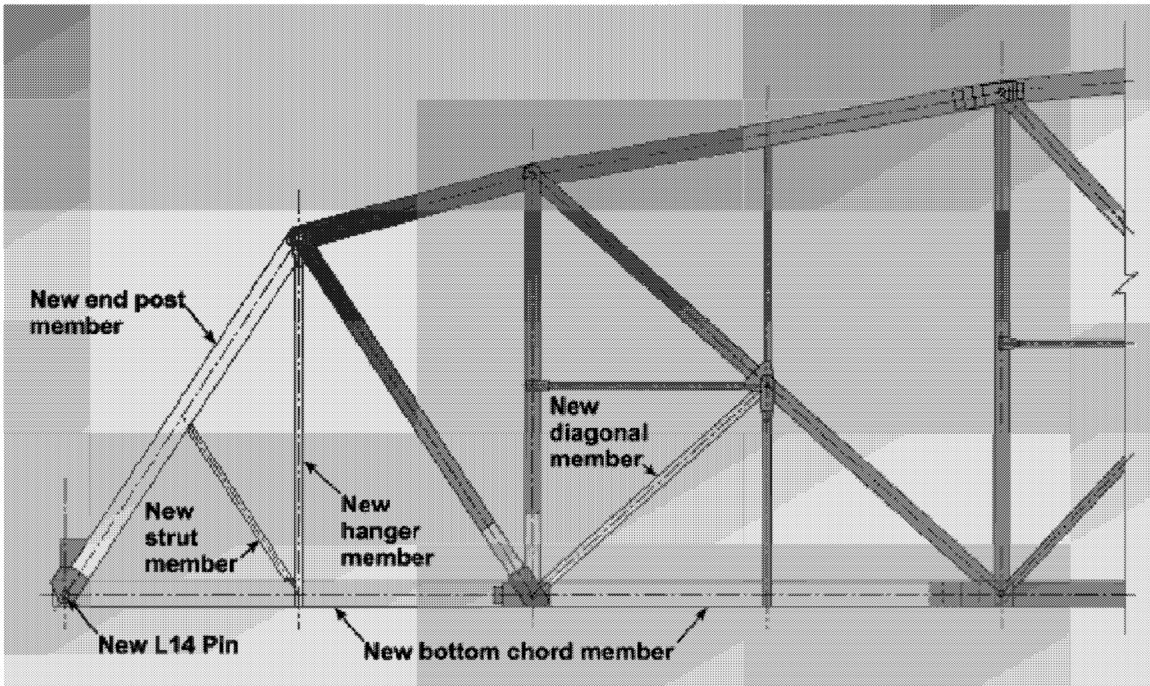

Figure 4, Rehabilitation of Span 4.

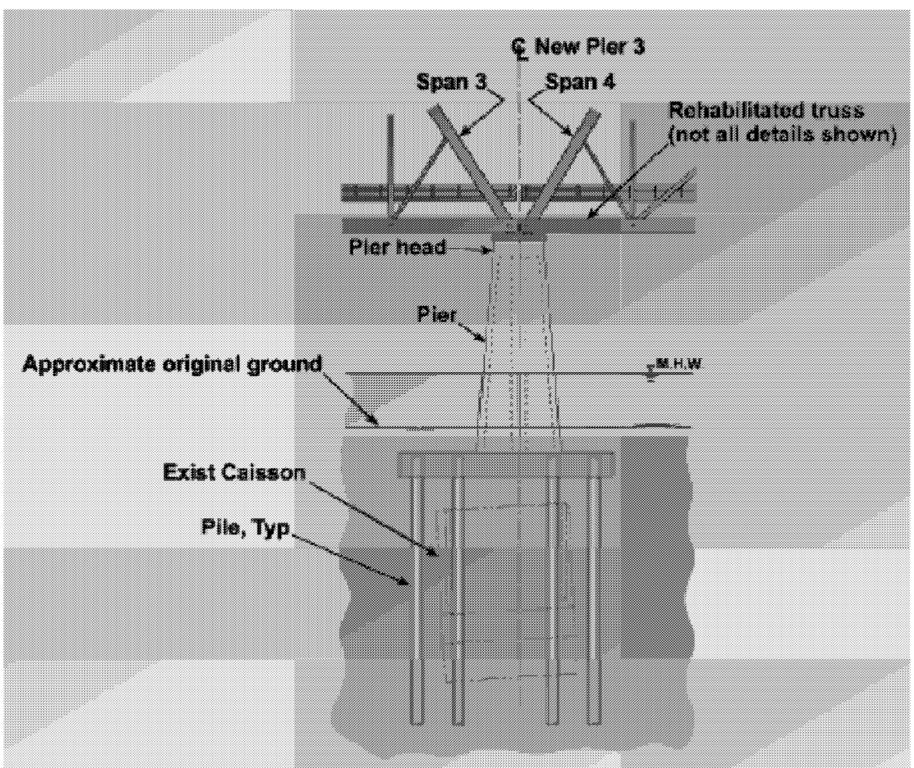

Figure 5, Replacement of Pier 3.

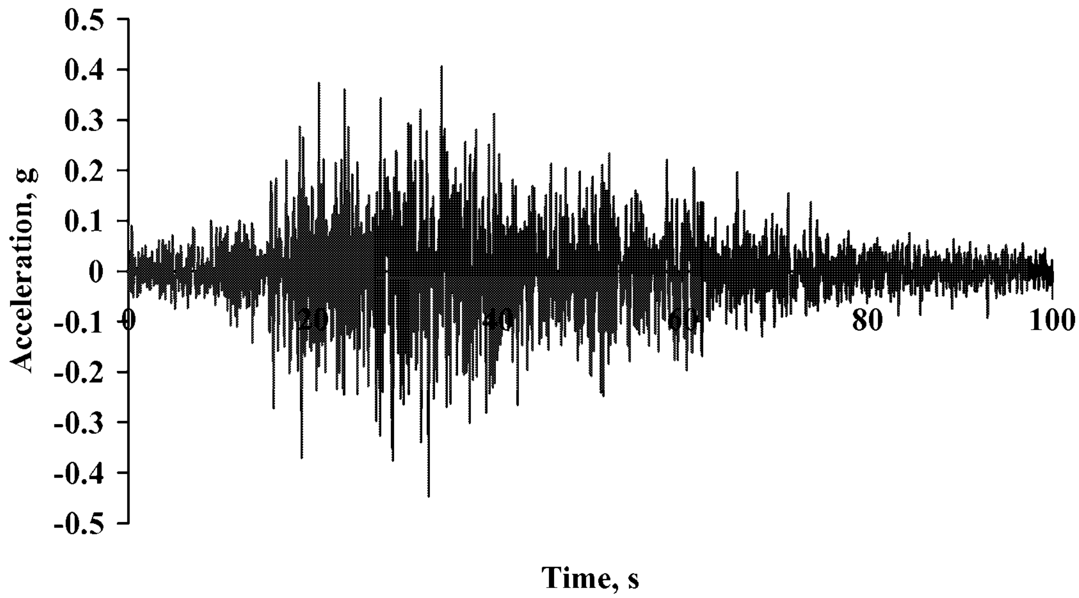

Figure 6: Ground Motion for Dynamic Analysis.

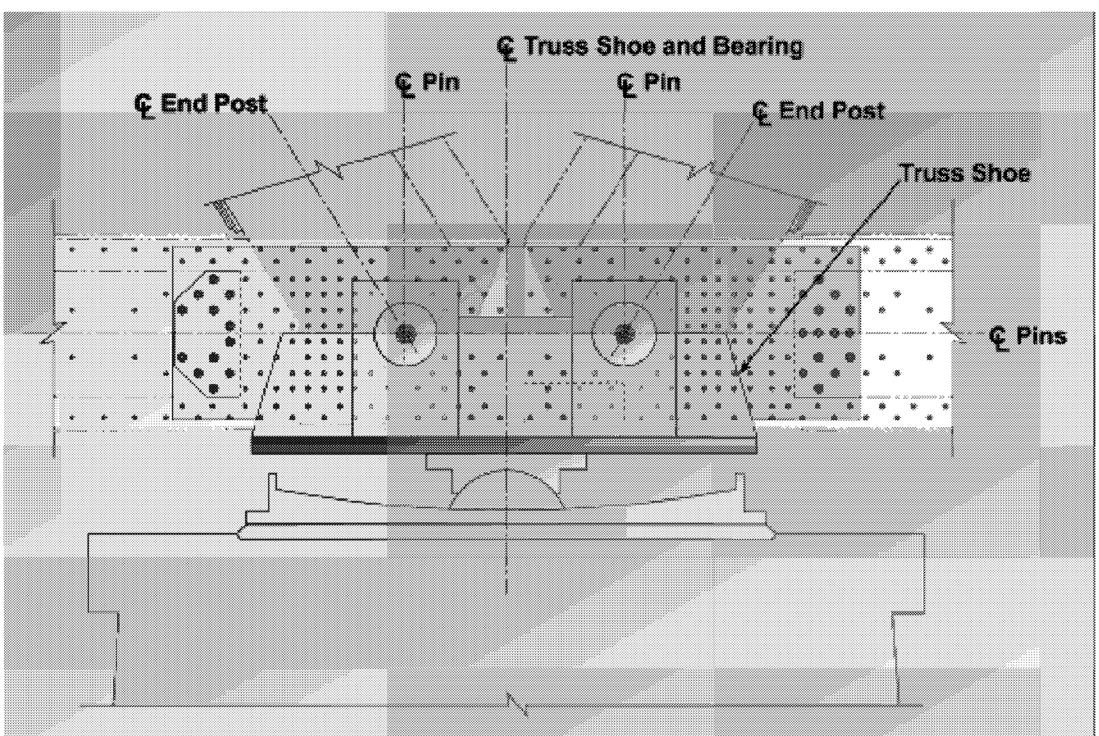

Figure 7, Installation of Friction Pendulum Bearings.

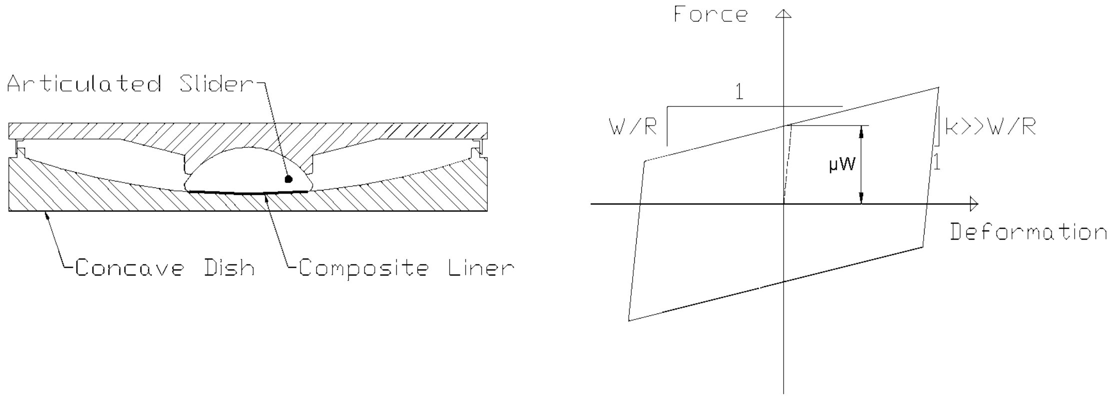

Figure 8: Friction Pendulum Bearing and Idealized Bilinear Hysteresis Loop.

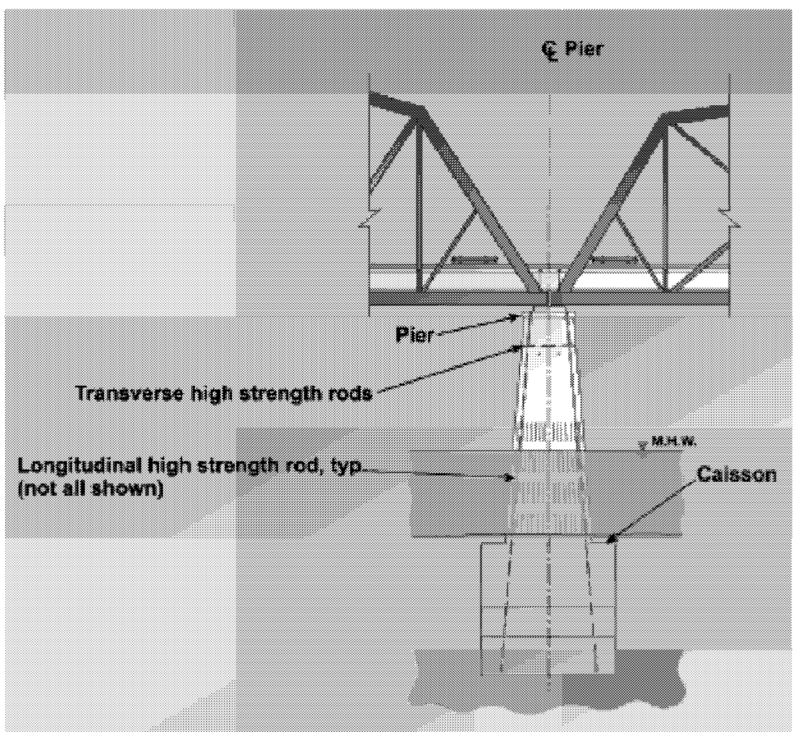

Figure 9, Retrofit of Piers 1 & 2.

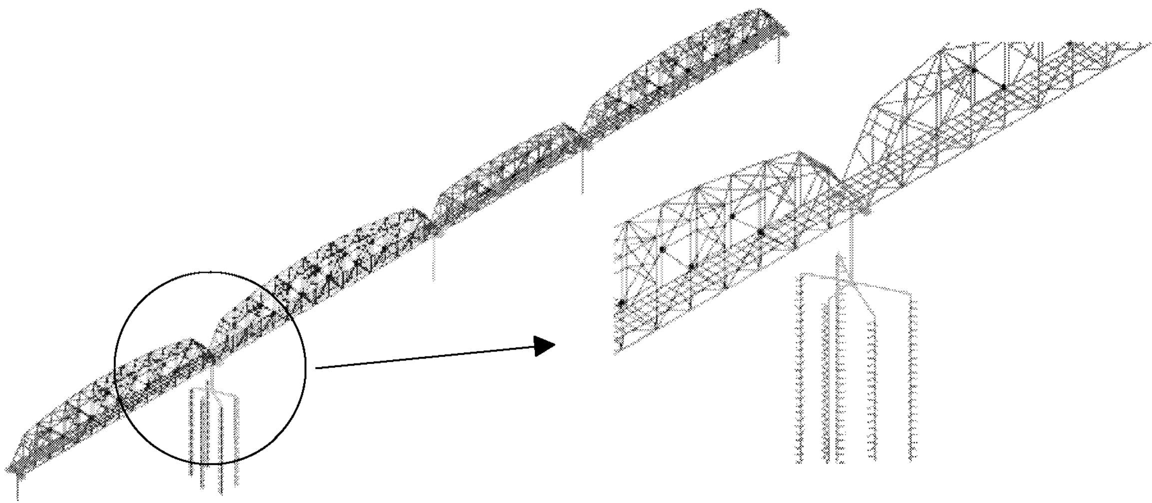

Figure 10: ADINA Model for Dynamic Analysis of the Bridge.

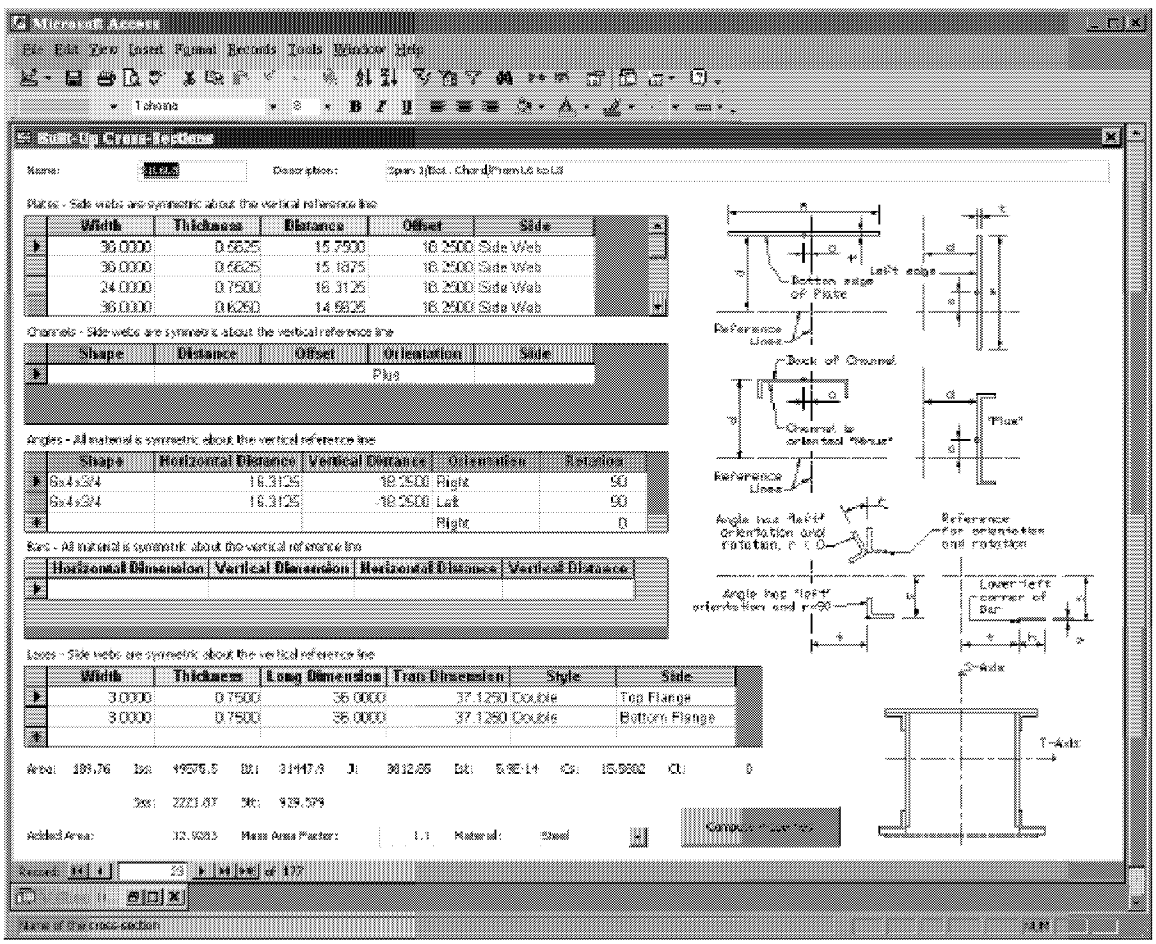

Figure 11: Screen Shot from Truss Database.

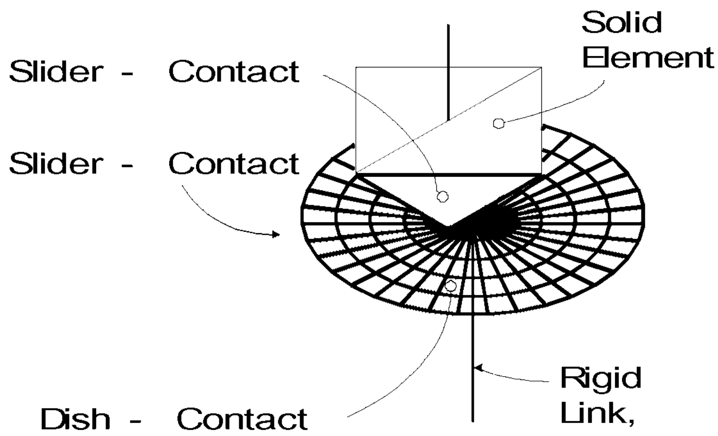

Figure 12: Contact Surface Model for Analysis of Friction Pendulum Bearings.

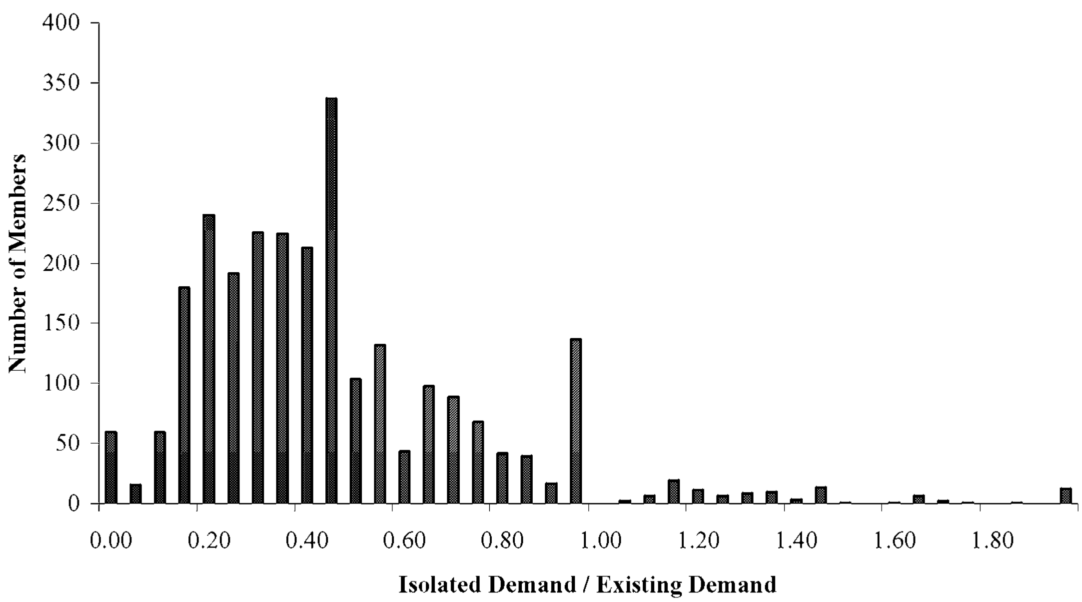

Figure 13: Reduction of Superstructure Forces through Isolation.

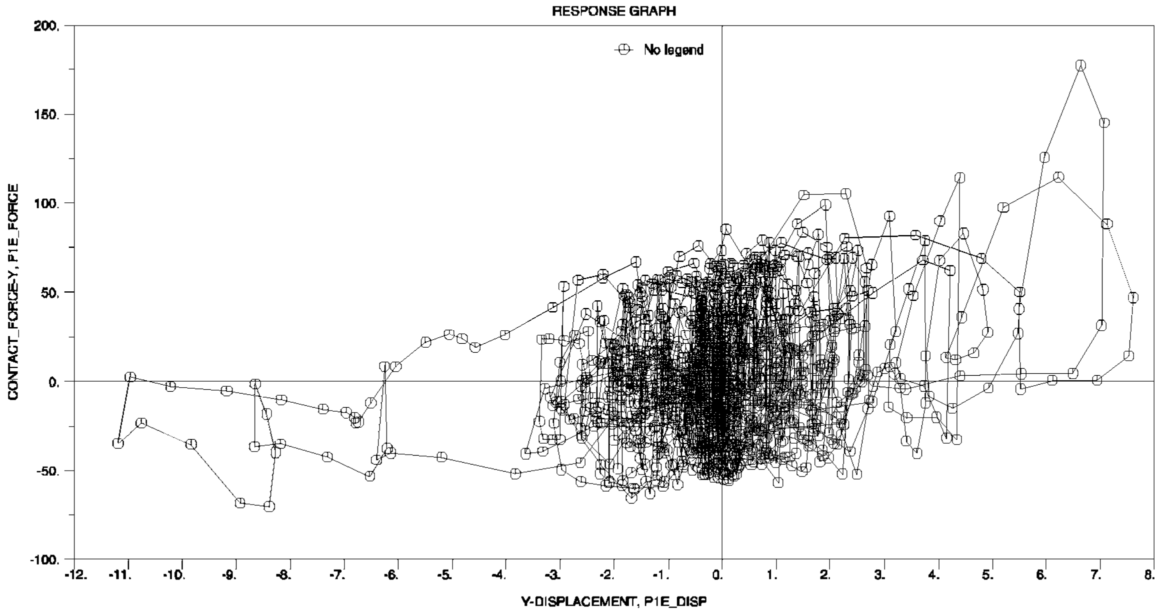

Figure 14: Friction Pendulum Bearing Hysteresis Loops.