INCORPORATING BUCKLING RESTRAINED BRACES (BRB)
AS PART OF THE
AUBURN-FORESTHILL BRIDGE SEISMIC RETROFIT

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ABSTRACT

The Foresthill Bridge was built by the Bureau of Reclamation in 1973. The 2,428 ft. long bridge is comprised of three spans and sits 730 ft. above the North Fork of the American River. The parabolic haunched deck truss bridge has fracture critical, high-strength steel members. Under the California Department of Transportation's Local Agency Seismic Retrofit Program, Placer County embarked on the seismic retrofit design of this bridge, which incorporates the use of large Buckling Restrained Braces to improve the performance of the bridge during large seismic events while not affecting the current daily performance of the structure.

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INTRODUCTION

The Auburn-Foresthill Bridge is a steel deck truss type bridge that links the towns of Auburn and Foresthill, California. The bridge was built by Kawasaki Heavy Industries for the Bureau of Reclamation and first opened to the public on September 3, (Labor Day) 1973. The bridge is currently owned and maintained by Placer County.

The 2,428 ft. long bridge is comprised of three spans (639 ft. – 862 ft. – 639 ft.) and sits 730 ft. above the North Fork of the American River and would have spanned the reservoir created by the proposed, but never constructed Auburn Dam. The center 502 feet of span two is a suspended span. The reinforced concrete bridge deck is composite and provides two 20 ft. wide roads, separated for the entire length of the truss spans by a 16 ft.-8 inch wide opening. The bridge was designed for a future median widening that could be accomplished by the installation of new stringers and a concrete slab.

Two slender, hourglass-shaped piers provide support for the main span. These piers are 403 ft. in height, 60 ft. wide at the top and bottom, and tapered to a 25 ft. wide neck. The main piers have hollow cores extending 207 ft. from the 85-ft. square base. The bridge truss depth at each end and at the center is 50 ft. and increases to 100 ft. at the main piers, thus forming a pleasing parabolic shaped bottom chord. A photo of the Foresthill Bridge is shown in Figure 1 below.

FIGURE 1. AERIAL VIEW OF FORESTHILL BRIDGE LOOKING NORTH

As part of the California Department of Transportation's Local Agency Seismic Retrofit Program, Placer County embarked on the seismic retrofit design of this bridge. This project has taken many of the concepts and design methodologies that were developed during the retrofits of the California Toll Bridge Program and then added more innovation. The project tasks included Seismic Assessment, Retrofit Strategy, and Final Design and Construction. The Design Team completed the Final Design (PS&E) in August 2009. The construction contract was awarded to Golden State Bridge in December 2010 with a bid of $ 58,374,849. The Hanna Group is providing project management/construction management services during construction, which is expected to be completed in early 2014.

DESIGN INFORMATION

Quincy Engineering, Inc. (QEI) developed the Auburn-Foresthill Bridge Seismic Retrofit Design Criteria as a living document that evolved along with the project. This concept also followed the design methodologies used on the California Toll Bridge retrofits, because of the unique features and aspects attributed to those structures. QEI's project specific design criteria followed a similar format of the major toll bridge crossing retrofit projects previously completed in California and was reviewed and approved by a Project Peer Review Panel and by Caltrans.

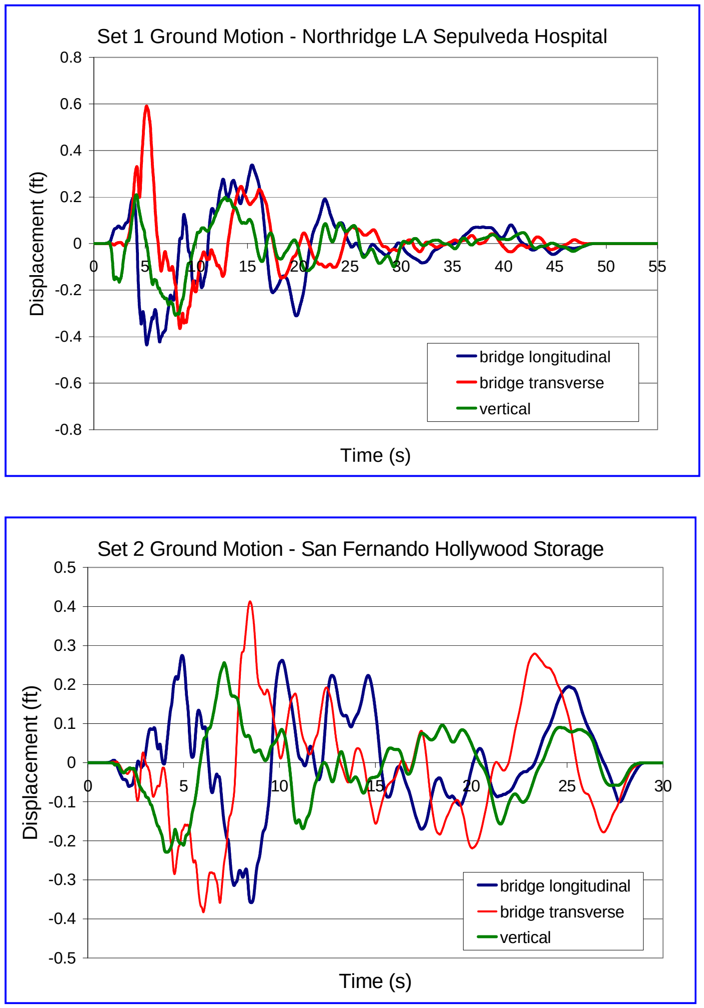


FIGURE 2. SEISMIC GROUND MOTIONS

The criteria included seismic performance requirements, loads and combinations, analysis methods, foundation design requirements, nonlinear foundation springs, steel and concrete material properties, as well as component design parameters.

The criteria documents the seismic hazards and the existing faults in the vicinity of the project. The criteria also discusses how the three ground motions were generated by modifying actual earthquake records with a spectrum matching technique such that the modified time histories would have the same shaking level as the ARS design curves adopted for the design.

The final design utilized the enveloped demands of all three motions as shown in Figure 2 to ensure the structure met the performance requirements. The performance goal was to limit seismic damage to the elements and locations that were inspectable and repairable to allow full service to be restored within months.

FINITE ELEMENT MODEL

A detailed three-dimensional model of the bridge was constructed by SC Solutions using the general purpose finite element program ADINA as shown in Figure 3. The finite element (FE) model included material and geometric nonlinearities. Nonlinear plastic beam elements (moment curvature elements) were used to simulate the behavior of superstructure and pier elements.

These nonlinear plastic beam elements captured nonlinear axial and bending member behavior, as well as global buckling of the superstructure elements. The concrete abutment structures were completely independent of the main bridge structure and were not included in the model.

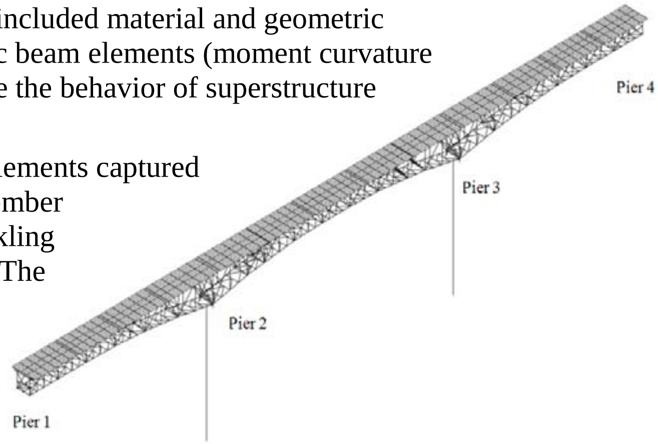


FIGURE 3. ADINA MODEL

At Piers 1 and 4 the bridge is resting on fixed vertical bearings and is anchored in the longitudinal direction with longitudinal anchors modeled with nonlinear elastic truss elements with calculated tensile and compressive capacities. In the retrofit model, buckling restrained braces (BRB) were added and modeled with nonlinear plastic elements with kinematic hardening. The fixed vertical bearings were modeled with nonlinear-elastic vertical-compression-only springs. The bridge was also restrained in the transverse direction by transverse anchors at Piers 1 and 4 in the vertical and transverse translational directions.

Input ground motions in the form of synchronous displacement time-histories were applied to the ground nodes at each boundary location. For retrofit, the transverse anchors were removed and replaced with two transverse keys modeled with non-linear spring elements. In the retrofit model, transverse ground motions at Piers 1 and 4 were applied only at the new transverse keys.

The existing truss bears on top of Piers 2 and 3 and was not restrained in longitudinal translation. The pier top rocker bearings were modeled explicitly with rigid compression-only vertical spring elements and rigid linear elastic transverse spring elements. The rocker bearings shown in Figure 4 allowed a maximum displacement of ±12 inches.

FIGURE 4. PIER 2 BEARING

In the as-built model, longitudinal translation was assumed to be unrestrained. For retrofit, longitudinal translation was assumed to be unrestrained up to a displacement of ± 11 inches, where a non-linear spring representing the friction/sliding of the bearing was engaged.

LONGITUDINAL ANCHOR RETROFIT

The truss superstructure is anchored to Piers 1 and 4 in the longitudinal direction with longitudinal anchors (as shown in Figure 5), and consists of two 2 inch x 12 inch ASTM A441 anchor plates embedded into the concrete foundation. The anchor plates are connected to each of the main truss gussets with two 1.50 inch x 18 inch ASTM A441 link plates and 6.0 inch diameter pins. The link plate capacity is stronger than the anchor plate capacity.

FIGURE 5. LONGITUDINAL ANCHOR

Time history results show that the anchor plates will experience forces exceeding their capacity. Without these anchors holding the bridge in the longitudinal direction, the superstructure may impact the abutment walls or drift away from the concrete seats during seismic loading, resulting in loss of vertical support. The capacity of the anchor plates used in the time history analysis limits the magnitude of the forces transmitted to the truss superstructure. Retrofit schemes that increase the capacity of the longitudinal anchor system could increase the force levels transmitted to the truss and thus the amount of required truss strengthening. Strain time history demands on the anchor plates show that the plates will have strains close to the ultimate strain limit of the steel members and failure of the members is possible. The stronger component, the link plates, remains elastic for the duration of the time history analysis.

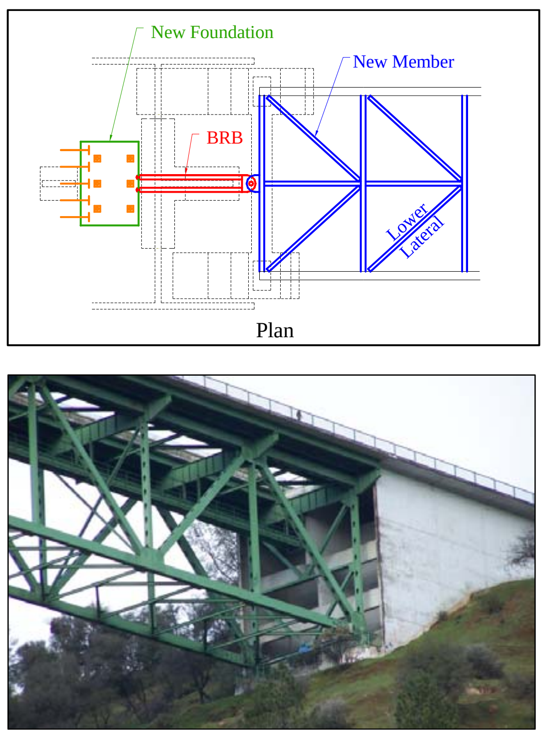


FIGURE 6. BRB LAYOUT AT PIER 1 AND 4.

To provide longitudinal stability after link plate yielding, a seismic response modification device, buckling restrained braces (BRB), was installed as shown in Figure 6 to ensure that forces transferred to the superstructure do not exceed acceptable levels while still providing longitudinal stability.

BRB have the advantage of providing a ductile system to restrain the longitudinal movement of the trusses while limiting the forces transmitted to the trusses. The devices are anchored to a new longitudinal strut at Piers 1 and 4, and to new separate foundations constructed under the approach spans. The BRBs are relatively maintenance free with only periodic painting required. They can be easily inspected after a seismic event and can be readily replaced if the BRB experiences large seismic strains. Longitudinal anchors are retrofitted by slightly reducing the capacity of the link plates to insure that yielding occurs in elements that can be inspected and repaired after a significant seismic event.

The retrofitted link plate shown in Figure 7 at right was specified to have controlled yield properties to closely define when the BRB's would be engaged.

The BRB system was installed at the centerline of the bridge so the truss is free to rotate about a vertical axis and forces from rotational restraint will not magnify the required longitudinal anchorage forces.

FIGURE 7. RETROFIT.

BUCKLING RESTRAINED BRACES

Buckling restrained braces are generally constructed of a cruciform or rectangular steel core surrounded by a debonding material and encased in a steel hollow tube filled with grout. The steel core carries the axial load while the outer tube, via the concrete, provides lateral support to the core and prevents global buckling. The core is free to yield in tension and compression.

Three BRB manufacturers that were known to manufacture BRB that could be utilized on this project were contacted to determine the design parameters for the time-history analysis and to make sure that design requirements could be met by the vendors. They included:

- Core Brace, West Jordan, UT
- Star Seismic,Park City, UT
- Nippon Steel Engineering Company, Tokyo, Japan

These manufacturers were extremely helpful in providing guidance on their manufacturing capabilities. Each manufacturer had a preferred end connection that included welded, bolted, or pinned connections and provided suggested connection details. The manufacturers also suggested that the strain should be limited to 2%. The manufacturer also provided guide specifications and suggested force tolerances for testing, since previous testing protocols for devices of this size and magnitude had not been developed.

The BRB were modeled in the non-linear time-history model using a non-linear plastic link element (with kinematic hardening) to model the yielding portion of the brace and with elastic end elements to model the non-yielding ends.   The following are the anticipated parameters of the BRBs as shown in Figures 8 and 9:

Yield Length = 171 inch	End Length = 12 inch
Yield Area = 48.0 in^2	End Area = 61.32 in^2
Yield Stress = ± 42 ksi	Yield Force = 2,000 kips

Tensile Plastic Stiffness = 3.5% Elastic Stiffness

Compressive Plastic Stiffness = 4.5% Elastic Stiffness

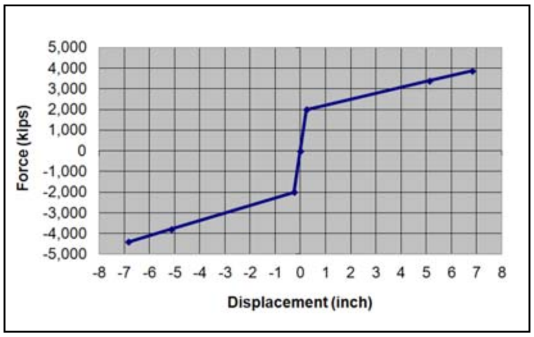


FIGURE 8. BRB FORCE VS. DISPLACEMENT.

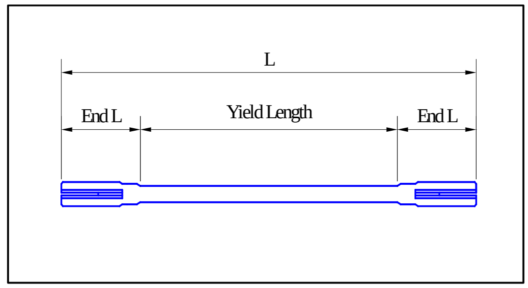


FIGURE 9. BRB YIELD LENGTH.

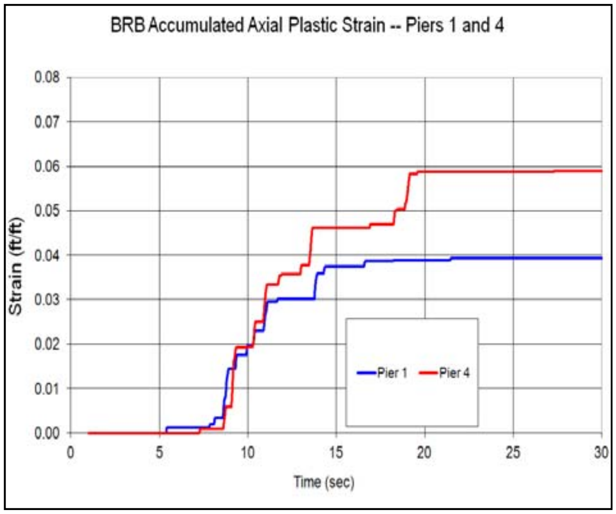
The longitudinal anchor elements are removed from the system when their strains exceed 4%. Following the rupture of all longitudinal anchors elements, longitudinal ground motions at Piers 1 and 4 are only applied through points where the BRB system anchors to the foundation. The lower lateral chevron bracing members near Piers 1 and 4 have strain demands exceeding criteria. The lower laterals were replaced with new members and a new longitudinal member was added to transfer the BRB forces into the bracing system as shown in Figure 6. The new members and the BRB connections have been designed to the BRB forces corresponding to 1.5 times the maximum BRB strains reported in the time-history analysis. The installation of the new bracing system required a detailed construction sequence so that the wind load resisting system of the structure remained intact at all times.

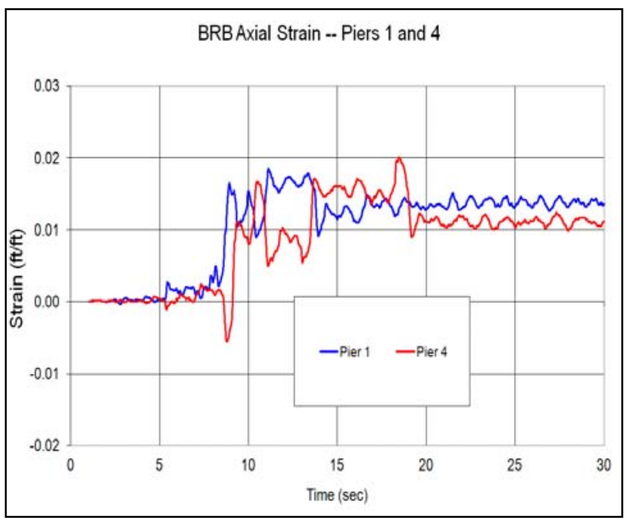
DESIGN PROCESS

During the design process SC Solutions provided time-history analysis results (as shown in Figure 10 on the next page) that included:

- Force-Displacement plots to provide a measure of the forces that would be required to be transferred;
- Time vs Strain plots to insure the strain did not exceed 2% as recommended by the manufacturers;
- Time vs Accumulated plastic strain to provide a measure of energy absorbed and to insure that the accumulated plastic strain did not exceed 200 times the yield strain; and

- Time vs Displacement plots of Relative Displacements between the Truss and Piers and Expansion Joint at Node U15 to insure that the available displacement capacities were not exceeded.





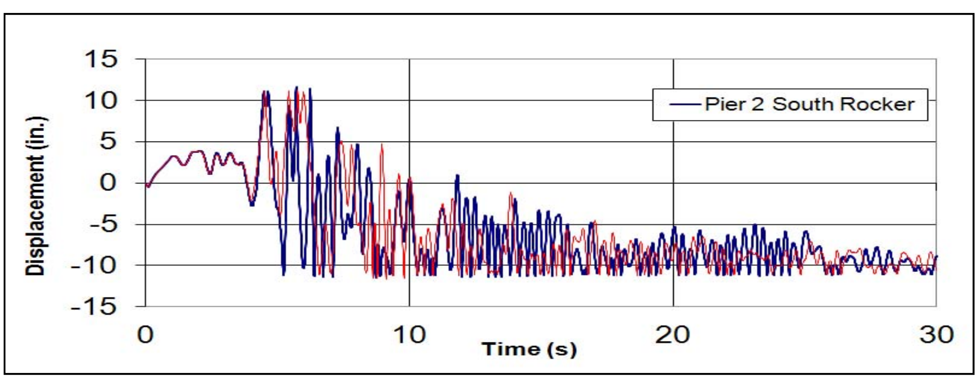


FIGURE 10. BRB MODEL RESULTS.

The yield capacity of the BRB's was kept to maximum of 1000 kips so that prototype testing could be accomplished within the limits of the UCSD facility, and the maximum strain was limited to 2% to keep within the limits of the BRB manufacturers. The yield length of the BRB used in the model was adjusted during the design process to limit the strain but was kept within the available length of specimens that could be tested at the UCSD facility. Ultimately, the testing limitations and protocol for these devices limited the size and magnitude of the BRBs.

PROJECT PLANS AND SPECIFICATIONS

The project plans defined the performance characteristics of the BRB in the form of a plot, as shown in Figure 11 below. Bolted or pin connections were permitted to accommodate the standards of various manufacturers, and both types of connections were designed and incorporated into the plans.

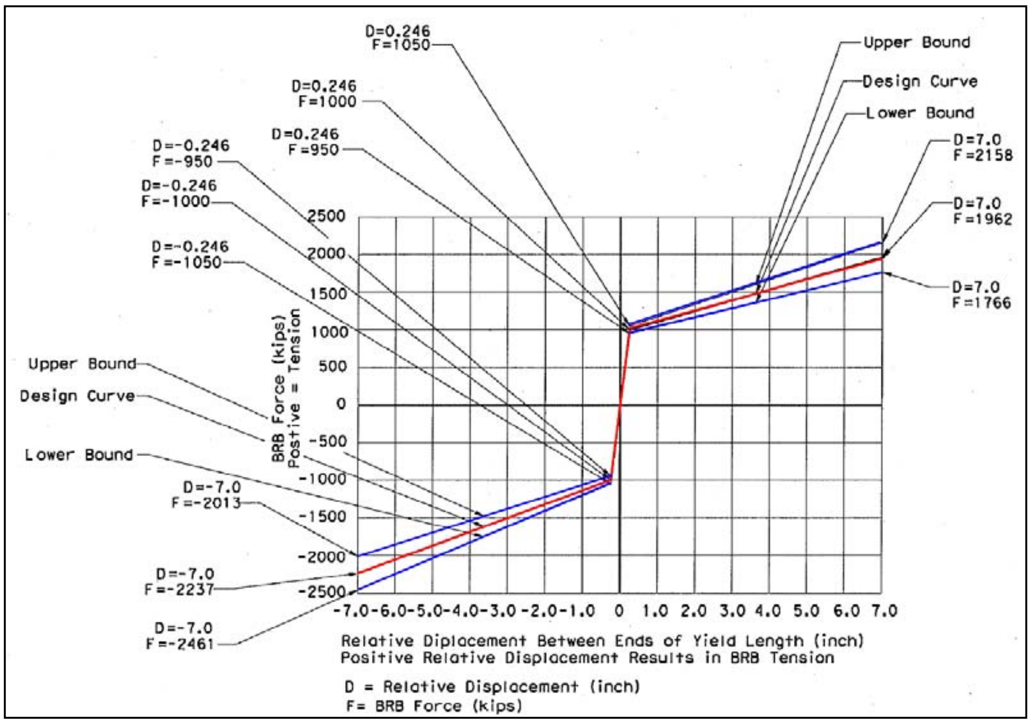


FIGURE 11. BRB CHARACTERISTIC CURVE.

BRB manufacturers bidding on this project were limited to those manufacturers that have successfully tested BRB's similar to the proposed BRB in the project plans. The contractor was allotted twenty-eight (28) weeks to produce working drawings and calculations, provide results of testing on two prototype specimens, and to produce final working drawings, a quality control program, and a maintenance manual for these devices. Stress-strain test results were required for the production of the prototype BRB's to be tested and the steel materials used for the BRB's shipped to the site were required to be fabricated from plates cast from the same heat used for the fabrication of the prototype BRB. Connections of the BRB to the brackets on the structure were required to be designed by the manufacturer for a force resulting from a displacement of 1.5 times the design displacement and were designed not to slip at the yield force.

BRB SUPPLIER

Golden State Bridge, the General Contractor that won the bid selected Nippon Steel Engineering Company, Tokyo, Japan to provide the BRB's for the project. The BRB's were manufactured by Yajima USA in Reno, Nevada and were provided at a cost of $90,378 for four braces to be installed, plus two braces required for prototype testing. The braces have an overall length of 276 inches and a yield length of 171 inches. The cruciform steel section was fabricated from ASTM A36 steel that was tested for stress-strain properties. The yielding section is housed in an outer tube fabricated from HSS 16x16x0.3125. The end connections utilized 16-1.25 inch diameter ASTM A490 bolts as shown in Figure 12 above.

FIGURE 12. BRB CONNECTION

PROTOTYPE TESTING

Testing was conducted by the University of California, San Diego at the Seismic Response Modification Devices (SRMD) Testing Facility in July 2011 as shown in Figure 13. The testing cost of $29,742 was paid for by the Contractor and the cost of the testing at UCSD was quoted in the project specifications. At the time that this project was advertised for bid, UCSD was the only facility capable of testing to the required demands; and the facility has previously tested BRB devices. The facility has limitations on the test length of BRB specimens, which influenced the selection of the yield length. Because of the numerous demands on this testing facility for other projects, scheduling an available time for testing, that met the needs of the Contractor, required constant communication with UCSD and the Contractor for over a year.

FIGURE 13. UCSD TESTING.

Both Caltrans whom had a lot of the lab time tied up with their experiments and UCSD were very cooperative in scheduling this testing around their other commitments.

Two specimens were tested in accordance with the project specifications that were based upon the AISC 341-05 protocol (as shown in Figure 14) except that the test displacements were 1.50 times the following design displacements*:*

- Axial Displacement
  = 3.42 inch (2% strain)
- Horizontal Axis Rotation
  = 0.003 radians
- Vertical Axis Rotation
  = 0.005 radians

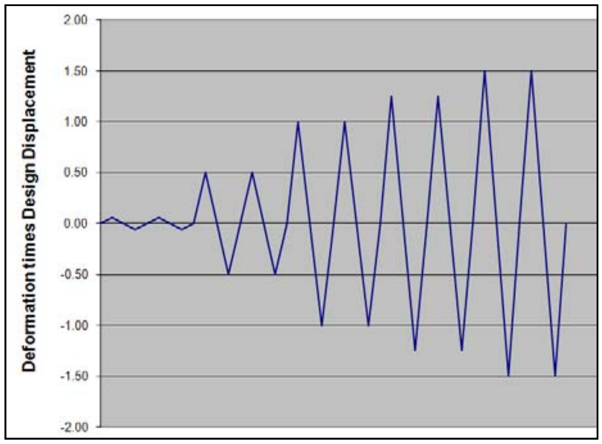


FIGURE 14. TESTING PROTOCOL.

The tests satisfied the following requirements, which were outlined in the project specifications:

- The two prototype tests shall display dynamic characteristics within 5% of each other.
- Test results shall display force-displacement characteristics within the upper and lower bound envelope shown on the plans. See Figure 15 below for results.
- The force-displacement hysteresis loops shall exhibit stable, repeatable behavior with positive stiffness and no pinched hysteretic behavior for all cycles.
- The BRB's shall show no signs of distress up to an inelastic axial deformation of 200 times the yield deformation. The tests results showed accumulative plastic strains of 828 and 1055 ε_Y.

The testing protocol required additional final cycles to failure, but for reasons of safety and difficulties in the performance of the testing machine, the specimens could not be tested to failure. The original intent was to test these devices to failure.

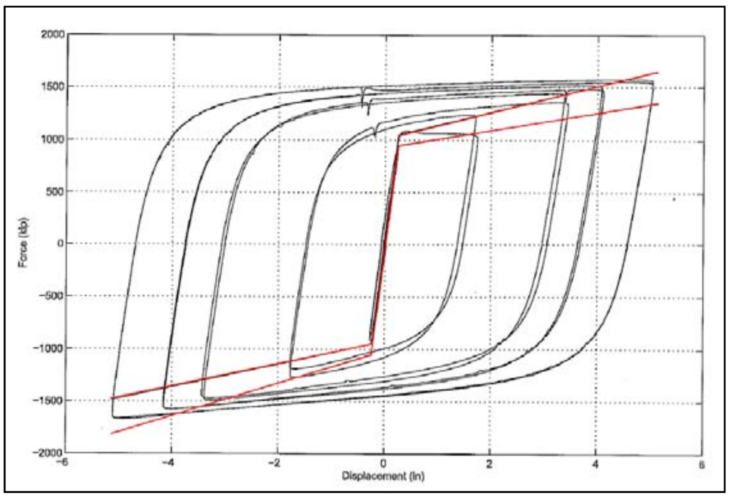


FIGURE 15. FORCE-DISPLACEMENT TEST RESULTS.

CONSTRUCTION

The erection of the BRB was similar to the installation of other structural steel members and did not involve any additional substantial equipment beyond what was already being utilized. In fact, the erection of the BRB did not amount to the largest hoists during this project. At this time, all of the structural retrofit work has been completed on the project. The BRBs at both ends of the bridge have already been placed. The only work that remains on the project is to complete installation of modular joint seal, as well complete the re-coating of the entire bridge. There were no significant construction change orders during the structural retrofit work including the BRBs. See Figure 16 for installation.

FIGURE 16. BRB INSTALLATION

CONCLUSION

The seismic retrofit of the Foresthill Bridge posed many challenges. These challenges were met by using the latest analysis tools and by innovative criteria and seismic modification devices. The BRB's allowed the designers to solve a stability issue with a passive device that allowed the service behavior of this truss to remain unchanged. The BRB's will not be activated until a large seismic event occurs, so that service level cycles will not impact the BRB. This eliminated any concerns that service loads and their affect on the bridge would be altered. Utilization of the BRB reduced a significant amount of additional steel retrofit of the truss and subsequent connections and thereby reduced the overall project costs substantially.

ACKNOWLEDGEMENTS

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The authors wish to acknowledge the contributions of the project sub-consultants, CH_2M Hill, SC Solutions, David Evans & Associates, Fugro, Earth Mechanics, Inc. and Cadre Design Group for their contributions to the project.

UNITS CONVERSION

1 inch = 25.4 mm	1 ksi = 6.895 megapascal
1 foot = 304.8 mm	1 kip = 4.448 kilonewton