SEISMIC EVALUATION AND RETROFIT OF SACRAMENTO RIVER BRIDGE AT RIO VISTA

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ABSTRACT

This paper discusses the seismic evaluation and retrofit for the Sacramento River Bridge at Rio Vista, California. Seismic evaluation studies for the 2500 ft long steel truss lift bridge consist of linear and nonlinear static and dynamic analyses. The analyses revealed that the most vulnerable elements in the bridge were the hinge and rocker bearings in the steel truss approach spans, pier bents and base connections for the tower columns. The retrofit of the bridge consists of seismic isolation of the bridge deck in the steel truss approach spans, and addition of passive energy dissipators at the tower column base connections. Extensive nonlinear dynamic time history analyses were performed to evaluate the performance of the isolation and energy dissipation systems.

INTRODUCTION

The Sacramento River Bridge at Rio Vista is a 2500 ft long, two lane steel truss bridge with a lift span. The structure was constructed in 1944 and 1958, and was awarded the AISC 'Most Beautiful Steel Bridge' award in 1960. The bridge may be subjected to a maximum credible earthquake of magnitude 6.75 from Coast ranges/Sierra Nevada Boundary Zone, with a PGA of 0.5 g. Seismic evaluation and retrofit studies were conducted to investigate the bridge for a 'No Collapse' criteria in a maximum credible seismic event and to develop a retrofit design. The seismic evaluation indicated that in the event of a major earthquake, typical hinge and rocker bearings in truss approach spans and base connections for tower columns could be severely damaged, leading to possible collapse of the structure. The retrofit for the steel truss

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approach spans consists of isolation of the bridge deck from the abutments and piers using seismic isolation bearings. Passive energy dissipators were added to the tower columns to control the excessive displacements and uplift caused by the rocking response of the towers. This paper describes the analyses conducted for seismic vulnerability studies, retrofit design, and evaluation of response of retrofitted structure.

DESCRIPTION OF EXISTING STRUCTURE

The Sacramento River Bridge at Rio Vista is a two lane bridge with east and west steel truss spans, towers and lift span, and a reinforced concrete slab bridge west approach. The east span of the bridge was built in 1944. The west span, towers, lift span and the west approach structure were built in 1958. The 2900 ft long structure consists of a 440 ft long west approach, a 633 ft long west span and a 1300 ft long east span. The lift span towers rise approximately 173 ft above the roadway deck and the lift span is 310 ft long. The bridge carries Route 12 over the Sacramento River and serves as an important link to Rio Vista.

![Figure 2 Bridge Elevation](image)

The west span consists of 4 spans, ranging from 140 ft to 210 ft in length. The east span consists of 8 spans, typically 180 ft in length. The bridge truss in the approach spans is typically supported by reinforced concrete piers which are founded on concrete and steel piles. At typical piers, the truss span is supported by hinge bearings, which are connected to the piers with four 1-1/2 inch diameter anchor bolts. At the abutments, the truss is typically supported by rocker bearings, which are anchored to the concrete with two 1-1/2 inch diameter anchor bolts.

Typical reinforced concrete pier bents consist of two circular columns connected together with a slender link beam (Figure 3). The sections are lightly reinforced in transverse direction and lack details for the effective confinement of concrete. Each column in a typical pier bent is supported by 19 steel H shaped piles (1944 construction) or 13 to 14 circular reinforced concrete piles (1958 construction). The piles typically extend up to approximately 40 ft into the soil. The towers are steel braced frames that rise approximately 173 ft above the roadway deck, typically supported on massive reinforced concrete piers (Figure 4). A 750 kip counterweight is provided at each tower as a part of the mechanism to raise

![Figure 3 Typical Approach Span Section](image)
and lower the lift span. Typical tower column is connected to a 9 ft square concrete block through eight 2-1/4 inch diameter bolts, embedded approximately 8 ft in the pier. The piers consist of four 9 ft square reinforced concrete legs, which are connected together by 2-1/2 ft thick concrete walls at the perimeter, and two 1 ft thick concrete walls in the interior, forming a hollow boxed section. The tower piers are founded on a grid of 145 precast concrete piles which typically extend approximately 60 ft into the soil.

SITE SPECIFIC SEISMIC HAZARD STUDIES

The surficial soils under the main bridge predominantly consist of loose to very loose sands, very soft to soft silts and clayey silts, extending to a depth of approximately 20 ft. Beneath the loose sands and soft clays, the soils typically consist of dense to very dense sands with gravels overlying very stiff to very hard clays to clayey silts reaching the bottom of borings. A target response spectrum (84 percentile) was developed based on a deterministic approach for a Maximum Credible Event ($M_w=6.75$) at the Coast Ranges/Sierra Nevada Boundary Zone (CRSNBJ) fault, which is located at a distance of 7 kms from the site. The spectrum corresponds to an event with approximately 1500 year return period. Three sets of baseline corrected ARS compatible time histories were generated based on spectrum matching algorithms. The time histories were based on 1994 Northridge earthquake (Arletta station), Hayward Event ($M_w=7-1/4$ at Richmond San Rafael Bridge) and 1989 Loma Prieta earthquake (South Street/Pine Drive, Hollister station).

SEISMIC EVALUATION OF EXISTING STRUCTURE

The Caltrans seismic performance criteria for this bridge was 'No Collapse' for the Maximum Credible Event. Considering the complexity of the bridge, comprehensive analyses of the structure were carried out using global and local models. The seismic evaluation consisted of a force and displacement based demand-to-capacity analysis.
Global force and displacement demands on the structure were computed using linear elastic dynamic response spectrum analysis. Typical pier bents were evaluated using static nonlinear lateral analysis. Tower frames were analyzed using static nonlinear lateral and dynamic nonlinear time history analyses. A description of the analyses and the vulnerability studies is as follows:

Global Analysis

A three dimensional finite element model of the bridge was developed in SAP90 V6.0 [2]. The model explicitly considered the spatial distribution of mass and stiffness in the structure (Figure 6). Foundation flexibility was considered by using translational and rotational springs at the base of pier column bents. Frame elements were used to model the individual members in the truss spans, the supporting piers, the towers, and the lift span. The tower columns, diaphragm floor beams at the deck level, portals, and the built up members at the top of the towers were modeled as beam-column elements. Since the counterweight is restrained by the rails attached to the side frames of the tower, 100 percent mass of the counterweight was assumed to contribute to the dynamic response of the towers.

![Figure 6 Global Analytical Model in SAP90](image)

Typical pier bents at truss spans were modeled as frame elements. Cracked section properties were used for reinforced concrete piers. The average cracked section properties were estimated to be about 1/2 the gross section properties. The tower piers were modeled as an assemblage of solid and shell elements. Three dimensional rotational and translational springs were typically provided at the base of each pier to model the stiffness of the pile group.

The as-built structure was analyzed for 100% Longitudinal + 30% Transverse response spectrum and 30% Longitudinal + 100% Transverse response spectrum for compression and tension models, using 50 Ritz vectors, with at least 95 percent mass participation. The main bridge structure essentially behaves as three different segments consisting of the west truss span, towers and lift span, and the east truss span. The mode shapes and time periods for the dominant modes of vibration in compression and tension models for these structural segments are shown in Figure 7. The transverse mode of the tower indicates a soft story mechanism.
Capacity evaluations were conducted for various structural components using actual or estimated material properties for existing construction materials in accordance with the provisions of AASHTO [1] and CALTRANS. Critical components were further studied using nonlinear static and dynamic analyses.

Mode 1, T=1.19 sec
East Span Longitudinal

Mode 7, T=0.88 sec
East Span Transverse

Mode 4, T=1.10 sec
Towers Longitudinal

Mode 2, T=1.15 sec
Towers Transverse

Figure 7 Vibration Mode Shapes

Pier Bent Analyses

Typical pier bent was evaluated with a static nonlinear lateral analysis to establish the yielding mechanisms and force-displacement behavior in both the longitudinal and the transverse directions. The piers were modeled as fiber hinge beam-column elements in DRAIN-3DX, with simplified material models for concrete and reinforcing steel [4]. The foundations were modeled as nonlinear springs to represent the rotational, lateral and vertical translation of the pile groups. In the transverse direction, the existing structure forms a mechanism by yielding of the pile group rotational springs, followed by flexural yielding of the link beam. However, shear failure of the link beam immediately follows the flexural yield (Figure 8). This mode of failure in the link beam is expected to be sudden and brittle and can lead to collapse of the pier bent.

Figure 8 Pier Bent Lateral Force-Displacement Behavior
Tower Frame Analyses

Static lateral nonlinear analyses of a two dimensional model of the as-built structure shows flexural yielding of anchor bolt bearing plates at low axial force demands, showing that the tower is likely to rock during a major earthquake. The rocking response of the tower frames at Pier 5 and Pier 6 was investigated with a three dimensional model in DRAIN-3DX [4] and ABAQUS [3]. The frame was modeled as linear elastic beam-column and truss elements as it was expected to stay essentially elastic for rocking response. Nonlinear dynamic time history analyses were conducted for three sets of site specific ground motion time histories, with simultaneous application of the horizontal and vertical components of the ground accelerations. The maximum displacements at the top of the tower reach approximately 60 inches, and the uplift reaches approximately 12 inches. Considering the high impact forces at the column base and possible lateral instability, retrofit of the base connections was required to limit the rocking response of the towers.

SEISMIC RETROFIT

The selected retrofit strategy consists of seismic isolation of east and west truss spans, and addition of passive energy dissipators at the typical base connections of tower columns. Seismic isolation of east and west truss spans incorporates replacement of existing hinge and rocker bearings with seismic isolation bearings. Passive energy dissipators are typically provided at tower column base connections to control the uplift during rocking response.

Seismic Isolation of Bridge Deck in Steel Truss Spans

The design constraints for the seismic isolation system consisted of force and displacements limitations imposed by the capacity of existing structure. The maximum forces on the pier bents were limited to prevent or minimize excessive rotation of the column foundation pile groups. The design displacements for the isolated deck were limited to less than 12 inches in the longitudinal direction, in order to prevent the pounding of the isolated deck against the tower columns which can result in severe damage to the tower frames. Three different isolation systems were considered including High Damping Rubber bearings, Friction Pendulum System bearings, and Lead Core Rubber bearings. Preliminary designs were developed using simple analytical models. Designs were based on high damping response spectra and considered the effects of deck flexibility in the transverse direction [5].

The performance of the isolation system was evaluated using three dimensional nonlinear time history analysis. Analytical models were developed for the east and west span in DRAIN-3DX. The substructure and superstructure were modeled considering the spatial distribution of mass and stiffness. Foundation flexibility was modeled with
nonlinear translational and rotational springs. The modeling of the bearings considered the nonlinear behavior and hysteresis characteristics of the devices. The isolated structure was analyzed for three sets of site specific ground acceleration time histories, with simultaneous application of the horizontal and the vertical components. The transverse and longitudinal displacement response of lead core rubber bearings at an intermediate pier is shown in Figure 10

![Figure 10 Response of Isolated Bridge Deck with Lead Core Rubber Bearings](image)

**Figure 10** Response of Isolated Bridge Deck with Lead Core Rubber Bearings

**Tower Frames**

Passive energy dissipation systems were used at the base of tower columns to control the rocking response of the tower frames (Figure 11). Systems studied included both rate independent and rate dependent devices. Preliminary designs were developed for a friction based device consisting of slotted bolted connections in a steel-on-brass friction couple, a Lead Extrusion Damper (LED), and a low exponent Viscous Damper (VD) [4]. Analytical studies showed both the LED and the low exponent VD to be very effective in controlling the rocking response of the tower, resulting in significant reduction in the uplift of tower columns. Three dimensional models for towers developed in DRAIN-3DX and ABAQUS were used for analytical studies of the energy dissipation systems. The structures were typically analyzed for three sets of site specific ground acceleration time histories, with simultaneous application of the horizontal and vertical components. The study indicated optimal performance for 100 kip yield force LED, with one device used at each column base in the tower frame. The uplift time histories, for both the existing and the retrofitted structure, for tower at pier 6 are shown in Figure 12.

![Figure 11 Passive Energy Dissipator at Tower Base](image)

**Figure 11** Passive Energy Dissipator at Tower Base

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CONCLUSIONS

Seismic evaluation and retrofit design studies were conducted for Sacramento River Bridge at Rio Vista. The bridge was evaluated and retrofitted for 'No Collapse' in a Maximum Credible Earthquake. Global and local analyses of the existing structure indicated that in the event of major earthquake, severe damage and possible collapse of the truss spans and the lift span towers could be expected. The seismic retrofit measures for the bridge includes seismic isolation of the east and west spans using seismic isolation bearings and addition of passive energy dissipators at the base of tower columns. Energy dissipation devices were shown to be very effective in controlling the rocking response of the tower frames with significant reduction in the uplift of column bases.

REFERENCES