SEISMIC RESPONSE OF HORIZONTALLY CURVED BRIDGES

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Abstract

To gain insight into the behavior of horizontally curved girder bridges and develop seismic design guidelines, a 2/5th -scale model of a highly curved bridge (overall length/radius = 1.8) has been studied experimentally using the shake table array at the University of Nevada Reno. Six different configurations of the bridge model were tested, so that the following comparisons of performance could be studied: with and without sacrificial shear keys at the abutments, with and without live load, with and without earthquake protective systems (full and hybrid isolation, ductile cross frames, and rocking piers); and with and without pounding at the abutments. This paper provides a brief summary of the project and the initial conclusions from testing each of the six configurations.

Introduction

This project is part of a larger project funded by the Federal Highway Administration (FHWA) that focuses on the seismic resilience of highway systems. To support this effort, analytical and experimental studies of the system performance of a three-span bridge with a high degree of curvature have been undertaken. The experimental studies were conducted on a 2/5th -scale model of horizontally curved bridge supported on four shake tables in the Large-Scale Structures Laboratory of the University of Nevada Reno. The following parameters were studied:

- Curvature
- Live load
- Seismic isolation and ductile cross frames
- Abutment-soil interaction, and
- Column rocking

This experimental work has been a collaborative effort among eight graduate students and three faculty members.

Bridge Properties

The prototype geometry for this study was based on the FHWA Seismic Design of Bridges Design Example No. 6 [BERGER/ABAM, 1996], which is a three-span continuous cast-in-place concrete box girder bridge. The prototype bridge consists of three spans: a middle span of 152.5 ft (46.48 m), and two equal end spans of 105 ft (32.00 m) measured along the centerline of the bridge. The width of the bridge was 30 ft (9.14

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m) and the radius of curvature was 200 ft (60.96 m) to the center line of the bridge. The subtended angle was 104°.

For this study, the superstructure of the bridge was modified from a concrete box girder to a set of steel plate girders and crossframes. The reasons for this change were three fold: (1) to reduce the weight of the superstructure to meet the payload capacities of the shake tables, (2) to remove the need for falsework during construction thus reducing the time and cost of construction of the model, and (3) to enable the study of ductile cross frames. The model was scaled to 2/5ths of the prototype and supported on three, two-degree-of-freedom shake tables and one, six-degree-of-freedom shake table, see Figures 1 and 2.

The 3-span continuous girders were pinned over the pier bent caps and free to slide in the tangential direction at the abutments. Sacrificial shear keys were provided in the radial direction at the abutments to limit the loads transmitted to the abutment and foundations. These keys were designed to fail at a level of shaking equal to 75% of the Design Earthquake. For the purpose of design, the bridge was assumed to be located on a rock site in Seismic Zone 3, and the 1000-year design response spectrum to be given by PGA= 0.47g, S_S= 1.14g, and S_1= 0.41g.

The model columns were 24-in (610 mm) diameter column with longitudinal and lateral reinforcement ratios of approximately 1% . This translated to using longitudinal reinforcement of 16 #5 bars connected with #3 spiral reinforcement pitched at 2 in (51 mm). The column had a concrete clear cover of 0.75 in (19 mm), and spiral diameter of 22.125 in (562 mm). With these properties determined, the column section was checked to determine capacity limits following the AASHTO Guide Specifications [AASHTO, 2007] by completing a moment-curvature analysis using XTRACT [ISS, 2012].

Experiment Basics

The superstructure was constructed in three parts in the fabrication yard and then assembled in the laboratory using bolted moment-connections at the dead load inflection points in the middle span. About 96 ton (855 kN) of added weight was mounted on the deck to satisfy similitude requirements bringing the total weight of the model to about 160 ton (1425 kN), see Figure 1.

The Sylmar record from the 1994 Northridge, California, earthquake was used to load the model. It was scaled by a factor of 0.475 to give S_1 = 0.41g, the one-second spectral acceleration of the Design Earthquake. Thus the Design Earthquake for this experiment was 0.475 x SYL. The loading protocol for the experiment comprised increasing levels of earthquake shaking to study the progression of damage. For the conventional and live load bridges, each bridge was subjected to a total of 10 experiments using ground motions ranging from 0.1 to 3.5 times the Design Earthquake. White noise was also applied to the bridge in the North-South and East-West directions between each earthquake motion for system identification purposes. The remaining four experiments also included motions from the El-Centro and Hachinohe earthquakes. For each level of input motion, the El-Centro, Hachinohe and Sylmar records were scaled to have the same spectral acceleration at 1.0 sec ($S_{1)}$.

Analytical Comparison

Comparisons of the analytical and experimental data have been made using SAP2000 [CSI, 2012]. The analytical models of the bridge system included a beam-plate system for the superstructure, such that the deck is modeled as shell elements, and the girders as frame elements, and a full 3D finite element model. In the case of the beam-plate model, the top and bottom of each column is modeled using a fiber hinge to represent the plastic hinge region. These models include the effects of strain penetration and strain rate loading effects but do not include P-delta effects in the columns. Initial investigations show good correlation for the maximum displacement in the system, as seen in Figure 3 for the conventional bridge.

Conventional

The conventional bridge provided a baseline for comparison [Levi et al., 2011, 2012]. The loading sequence used for the bridge is shown in Table 1. After each motion, the bridge was checked for damage. The superstructure was constrained at both abutments in the radial direction by a shear key, but was free in the tangential direction. The key was designed to fail at a load of 25 kip (111 kN). This occurred at the 75% design earthquake. The experiment showed that the horizontal curvature of the bridge directly impacts the torsional loading and rotation at the columns and bearings while the keys are intact. Once they fail, torsional loading and rotation change as system torsional modes are excited by the higher amplitudes of ground motion. Along with these findings, reverse bending of the columns occurred and should be considered when designing single column bents in curved bridges. Figure 4 shows the damage that occurred in the columns after 350% of the design level earthquake, while Figure 5 shows the orbit of column displacement at 350%.

Live Load

To study the impact of live load, it was decided to place six loaded trucks on the bridge [Wibowo et al., 2012]. This was done after replacing the columns that were damaged during the conventional bridge experiments. The starting point for selection of the test vehicle was the H-20 truck from the Caltrans Bridge Design Specifications. This truck is a two-axle vehicle weighing 40 kip (178 kN) (8 kip (36 kN) on the front axle and 32 kip (142 kN) on the rear axle) with a 14-ft (4300-mm) wheel base. For a 2/5th -scale model, the model truck would have a wheel base of 5.6 ft (1700 mm), be 2.4 ft (730 mm) wide, and weigh 6.4 kip (28 kN). Since such a vehicle would most likely have to be custom-built, the decision was made to select from commercially available vehicles. The closest possible vehicle to match the modeling requirements was found to be the Ford F-250. The bridge weighed 320 kip (1400 kN)) without the trucks present, and 380 kip (1700 kN) with the trucks. Figure 6 shows the bridge with the trucks in place.

In low amplitude motions, when the shear keys were still intact, live load gave a beneficial effect (lower abutment shears, smaller column drifts, and less concrete spalling in the plastic hinge zones). In higher amplitude motions, after the shear keys had failed and the abutments were free to move, the effect due to live load was less significant. This may be due to (1) the deteriorating nature of the bridge under increasing levels of shaking and thus a changing vehicle-to-bridge frequency ratio, or (2) the changed configuration of the bridge when the abutments were released after the radial shear keys failed, or (3) both.

Full and Hybrid Isolation

The curved bridge was also tested for two different configurations of seismic protective systems [Monzon et al., 2012]. In the first configuration isolators are provided at all supports ("full isolation"), while in the second, isolators are only provided at the abutments ("hybrid isolation"). The isolators (lead-rubber bearings) in the first case were designed to keep the columns elastic under the design earthquake and to provide sufficient period shift and energy dissipation such that the columns remain essentially elastic even at maximum considered earthquake (150% of the design). The isolators in the second case were designed to not only keep the columns elastic but also substantially reduce the superstructure displacements. These isolators are larger than those in the first case and act as energy dissipators, attracting loads away from the columns to the abutments. Buckling restrained braces were added in the hybrid isolation case to limit the radial abutment shear forces by yielding.

It was shown that both isolation techniques are effective in keeping the columns essentially elastic even under the maximum considered earthquake. Figure 7 shows the columns after subjecting the bridge to earthquake level equal to 300% of the design. It was also shown that the hybrid isolation technique reduced the displacement by a factor of about two, but with a corresponding increase in the longitudinal abutment shear force. The smaller displacement however reduces the possibility of bridge pounding at the abutment backwalls and requires significantly smaller movement joints.

Abutment Backwall Interaction

In the abutment backwall case, the conventional bridge configuration was used but with a new set of columns and the addition of backwalls at both ends of the bridge, supported on seat-type abutments [Wieser et al., 2012]. In many bridge systems, the gap between the superstructure and backwall at a seat-type support allows for shrinkage and thermal movements to occur, but during a large seismic event, there is a possibility that this gap will close and high-acceleration seismic pounding will occur. Therefore, it is important to study the influence that abutment backwalls can have on system response.

The model backwall was reinforced concrete and spanned the width of the bridge. A gap of 1.875 in (48 mm) was determined from temperature load analysis of the prototype, and thus a scaled gap of 0.75 in (19 mm) was used in the model. The backwall was supported on two casters which travel along a rail oriented in the tangential direction of the bridge. The rails restrain movement of the backwall in the radial direction. Figure 8 shows the backwall system. The girder and deck of the bridge is shown on the left of this figure. The square plate at the bottom is the slider plate that allows translational movement of the bridge. The vertical element in the center of the figure is the reinforced concrete backwall with a steel frame. The mechanism on the right side is a set of nonlinear compression-only springs. These springs were developed to model the passive resistance of the soil backfill and comprise pre-deformed steel plates which yield in a buckled shape as the backwall is pushed back into the 'fill'.

The experiment demonstrated the influence that backwall pounding has on system response. Pounding limited the displacements of the bridge and reduced the damage in the column but generated increased deck accelerations, particularly near the ends of the bridge. In addition, significant radial friction forces were generated whenever the end of the superstructure was in contact with the backwall.

<u>Rocking</u>

In all of the bridge configurations discussed above, the columns were connected to rigid footings and post-tensioned to the shake tables. In order to investigate alternative ways to reduce damage, this base fixity was removed and the columns allowed to rock during the course of the experiment [Saad et al., 2012]. The footings were placed on rubber pads to simulate the soil stiffness. The dimensions of these pads were 8 in x 8 in x 2 in (203 mm x 203 mm x 51 mm) with a durometer of 44 which is equivalent to a modulus of elasticity of around 300 psi (2.07 MPa). The final stiffness of each pad was 32.0 kip/in (5.60 kN/mm) which resulted in a soil stiffness of 320 psi/in (87 kPa/mm) for the soil underneath the footing. The pier setup is shown in Fig. 9

Preliminary observations of the experiment indicated that rocking took place and the footing corners uplifted by about 2 in (51 mm) in some cases. This resulted in a behavior similar to that of an isolated bridge with lower column forces. There was however some damage to the columns, which provided energy dissipation. Slight spalling of concrete occurred in the top of the column plastic hinges, which was expected as the rotation of the bridge pier due to rocking increased the rotation in the top plastic hinges. There were increased levels of uplift at the abutments due to the footing rocking.

Conclusions

The shake table array at the University of Nevada, Reno has enabled a very unique project to be undertaken, which involved the experimental investigation of the system-effects in a horizontally curved bridge during an earthquake. The project included the study of a base line conventional bridge and then changed different parameters to determine their impact on system response. The project was able to demonstrate that current design methods work well and provide life safety. It also showed that seismic isolation, hybrid isolation, abutment pounding and column rocking reduce the level of damage in the columns to varying degrees, when compared to the performance of a conventional bridge. The project also showed that the influence of live load was beneficial for this particular bridge, but the benefit diminished with increasing earthquake ground motion.

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Table 1: Conventional Bridge Test Protocol

Date	Test Number	Test Name
7/26/2011	1	Run_01_0.1xDesign
	2	Run_02_0.2xDesign
	3	Run_03_0.5xDesign
7/29/2011	4	Run_04_0.75xDesign
	5	Run_05_1.0xDesign
	6	Run_06_1.5xDesign
8/2/2011	7	Run_07_2.0xDesign
	8	Run_08_2.5xDesign
	9	Run_09_3.0xDesign
	10	Run_10_3.5xDesign

Figure 1: Conventional model bridge

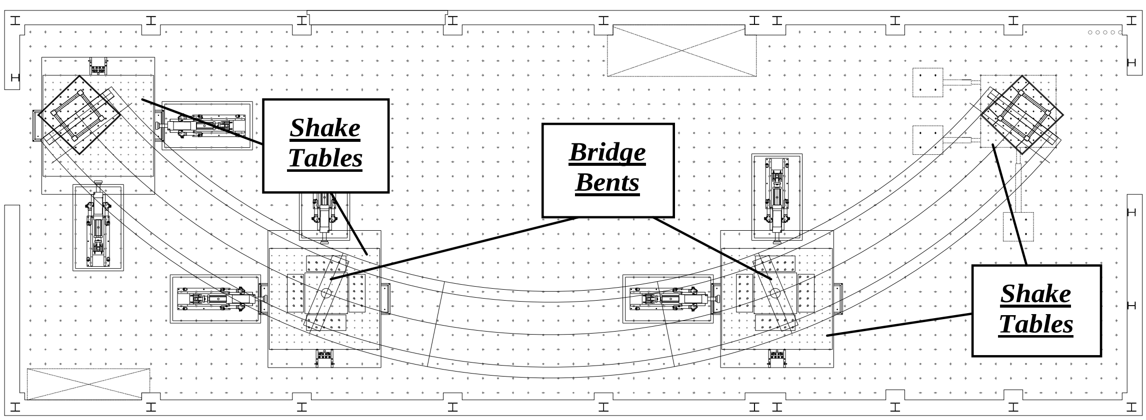


Figure 2: Bridge layout

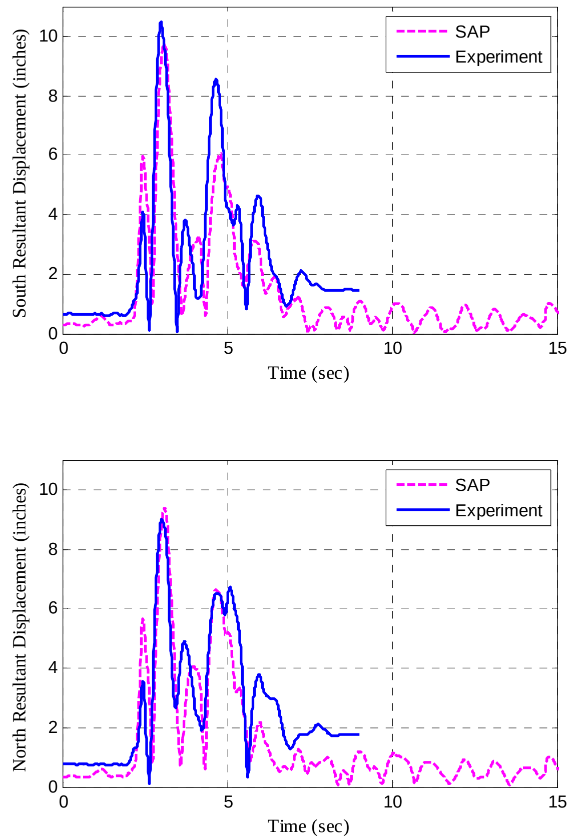


Figure 3: Convention bridge resultant displacement comparison @ 350% design earthquake (1 in = 25.4 mm)

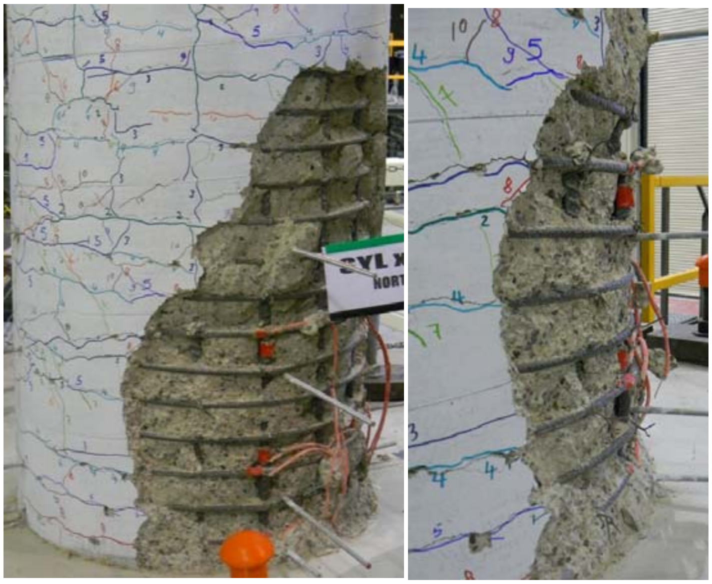


Figure 4: South bent @ 350% of design earthquake

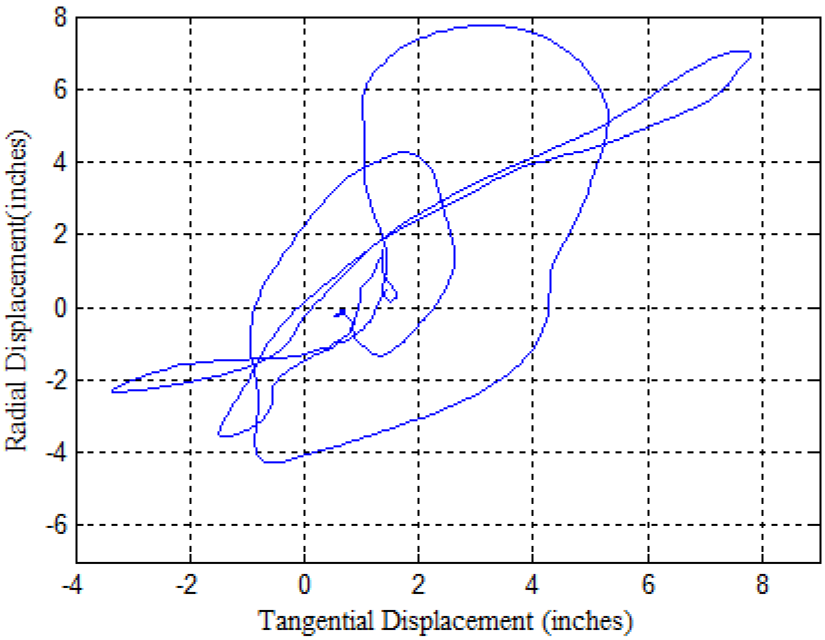
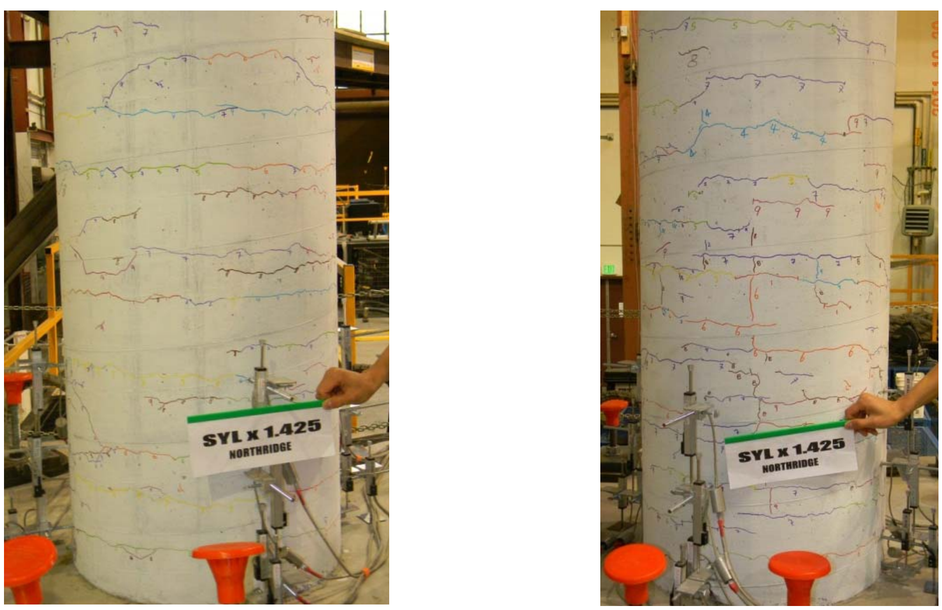


Figure 5: South bent orbital displacement @ 350% design earthquake (1 in =35.4 mm)

Figure 6: Bridge model with live load (Courtesy of M. Wolterbeek, 2011)



Bent 2 column   Bent 3 column

Figure 7: Full isolation columns after subjecting the bridge to 300% of design

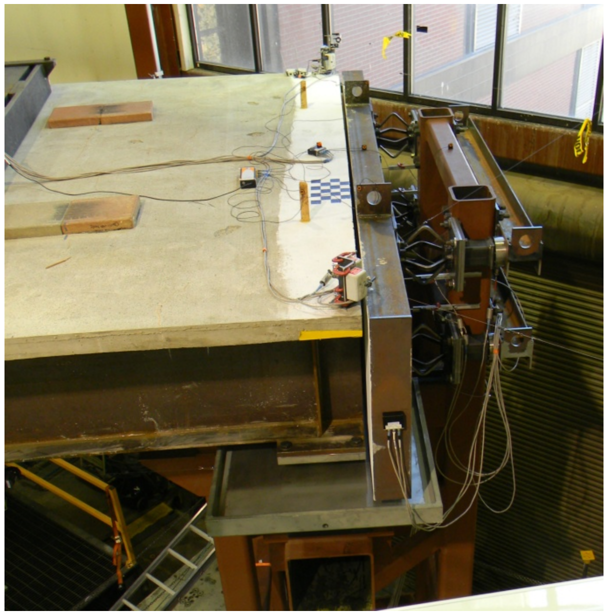


Figure 8: Radial view of backwall system

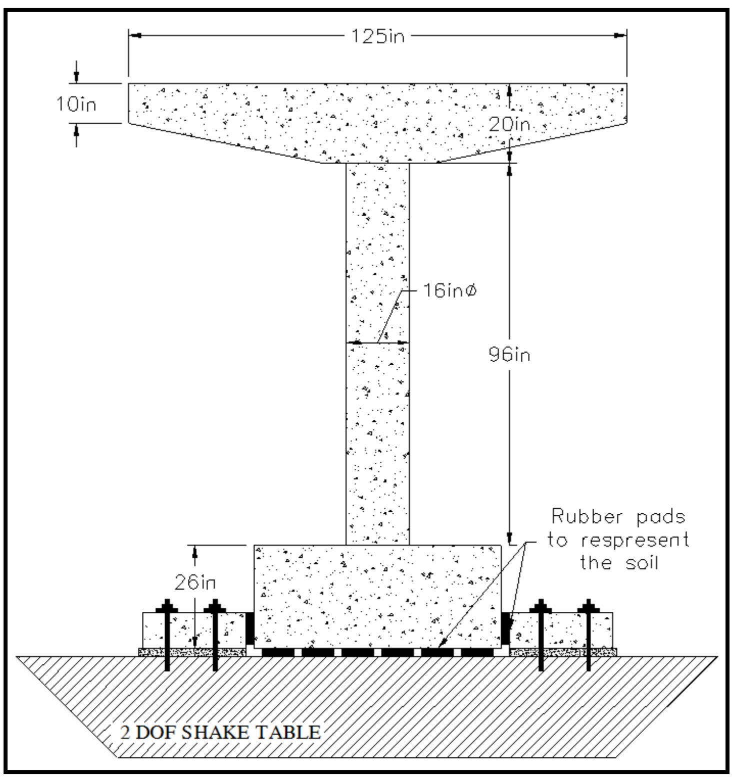


Figure 9: Experimental setup for the bridge bent (1 in = 25.4 mm)