Seismic Response of Columns in Horizontally Curved Bridges

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SUMMARY:
There are many components of a curved bridge system that influence behavior during seismic loading. The substructure, superstructure, and bearing types have a dramatic impact on system behavior. To study this impact, a 2/5ths scale, curved bridge was designed and tested on the multiple shake table array in the University of Nevada Reno, Large-Scale Structures Laboratory. In addition, extensive analytical modeling has been undertaken.

This paper focuses on the design and performance of the single-column, dropped-bent cap, substructure for the bridge. A parametric study was conducted to determine the optimal column size based on conventional details typically used in practice. After completing this study, the column was designed according to the AASHTO LRFD Bridge Design Specifications and AASHTO Seismic Guide Specifications for LRFD Seismic Bridge Design.

Specific elements of the substructure response were found to be determined by the global response of the system. These include the torsional stiffness of the system, abutment–column interaction, and column plastic hinging. In particular it was found that column behavior was a combination of single and double curvature despite being pinned to the superstructure. This was due to the constraints at the top and bottom of the column in the radial and tangential directions.

This paper discusses the performance of the columns during the experiment and presents initial comparisons with analytical results. It was concluded that the curvature of the bridge directly impacts the torsional loading and rotations at columns and bearings when shear keys are still intact. Once the shear keys fail, torsional loading and rotations change as system torsional modes are excited. Along with these discoveries, reverse bending of the columns is also expected and should be considered when designing single column bents in curved bridges. In addition, significant uplift of the girders occurred at large ground excitations and should be considered in the design process. Experimental column investigation in curved bridge systems has provided information that could not be determined from component testing.

Keywords: Curved Bridges, Large Scale, Experimental Testing

1. INTRODUCTION

The research reported in this paper is part of a larger project funded by the Federal Highway Administration (FHWA) on the seismic resilience of highway systems. Under this project, fragility functions and seismic design guidelines for horizontally curved highway bridges are being developed, and to support this effort, analytical and experimental studies of a three-span bridge with a high degree of curvature are being undertaken. In particular, system performance studies are being conducted on a scale model of this bridge supported on multiple shake tables in the Large-Scale Structures Laboratory and the effects of the following parameters on the response of the bridge are being studied:

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

This project is therefore a collaborative effort among eight graduate students and three faculty members. This paper discusses the performance of the columns during the experiment and presents initial comparisons with analytical results. More detail can be found in Levi & Sanders, 2011.

2. BRIDGE PROPERTIES

The prototype geometry for this study was based on the FHWA Seismic Design of Bridges Design Example No. 6 (BERGER/ABAM Engineers Inc., 1996), 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 (46.48m), and two equal end spans of 105 ft (32m) measured along the centerline of the bridge. The width of the bridge was 30 ft (9.14m) and the radius of curvature was 200 ft (60.96m) 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 three steel plate girders with crossframes. The reasons for this change were twofold: (1) to reduce the weight of the superstructure to meet the payload capacities of the shake tables and (2) to remove the need for falsework thus reducing both the time and cost of construction of the model. 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.

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.

3. COLUMN PROPERTIES

To determine the optimal column size, a parametric study was completed. In this parametric study, various column sizes and longitudinal reinforcement ratios were investigated. These parameters included diameters of 16 in (406.4 mm), 20 in (508.0 mm), and 24 in (609.6 mm). The longitudinal reinforcement ratios varied from 1% to 2% with a transverse reinforcement ratio of 1%.

From the parametric study, it was determined that a 24-inch (609.6 mm) column with longitudinal and lateral reinforcement ratios of approximately 1% had optimal properties for the set of experiments to be conducted. This translated to using longitudinal reinforcement of 16 #5 bars (1.10%) connected with #3 spiral reinforcement pitched at 2 in (50.8 mm) (0.99%). The column had a concrete clear cover of 0.75 in (19.1mm), and spiral diameter of 22.125 in (562.0 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 (XTRACT). The moment-curvature, shear, and ductility results are shown in Table 3.1 and 3.2.

Table 3.1. Moment-Curvature Results

Variable	Value	Units
Constant Load	577.6	kN
Curvature at First Yield	6.14E-04	1/mm
Effective Yield Curvature	8.66E-04	1/mm
Ultimate Curvature	1.73E-02	1/mm
Moment at First Yield	52893	kN-m
Effective Yield Moment	74533	kN-m
Ultimate Moment	80768	kN-m

Table 3.2. Column Shear and Ductility

$\varphi_s V_n$ (kN)	3607.7
V_u (kN)*	2236.0
$\varphi_s V_n/V_u$	1.61
V_n (kN)	4244.3
V_c (kN)	781.4
V_s (kN)	3743.4

Δ_{demand} (mm)	43.9
$\Delta_{capacity\ single}$ (mm)	176.1
$\Delta_{capacity\ double}$ (mm)	121.1
$\mu c_{single\ curvature}$	8.4
$\mu c_{double\ curvature}$	14.1

* Ultimate shear calculated from overstrength moment

4. EXPERIMENT

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. The splice connections were designed to make the girders and slabs continuous. About 96 tons (855 kN) of added weight was mounted on the deck to satisfy similitude requirements bringing the total weight of the model to about 160 tons (1425 kN), Figure 4.1.

Figure 4.1. Instrumented Model Bridge

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, A total of 10 tests were conducted using ground motions ranging from 0.1 to 3.5 times the

Design Earthquake, Table 4.1 . Low level white noise was also applied to the bridge in the North-South and East-West directions between each earthquake motion for system identification purposes.

Table 4.1. Testing 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

After each ground motion, the columns were investigated for damage. Yielding of some of the longitudinal reinforcement occurred at 75% of the Design Earthquake accompanied by cracking in the cover concrete. Yielding of all of longitudinal reinforcement occurred at the column-footing interface at 100% of the Design Earthquake but no spalling was observed. At 350% of the Design Earthquake, buckling of longitudinal reinforcement occurred in both columns as shown in Figure 4.2 & 4.3, Despite the occurrence of buckling, the lateral load capacity of the bridge had not begun to decrease as shown in the load-displacement hysteresis loops in Figure 4.4. If the maximum displacement is defined as the displacement at which the lateral load capacity drops to 80% of the maximum shear, these columns still have displacement capacity in reserve.

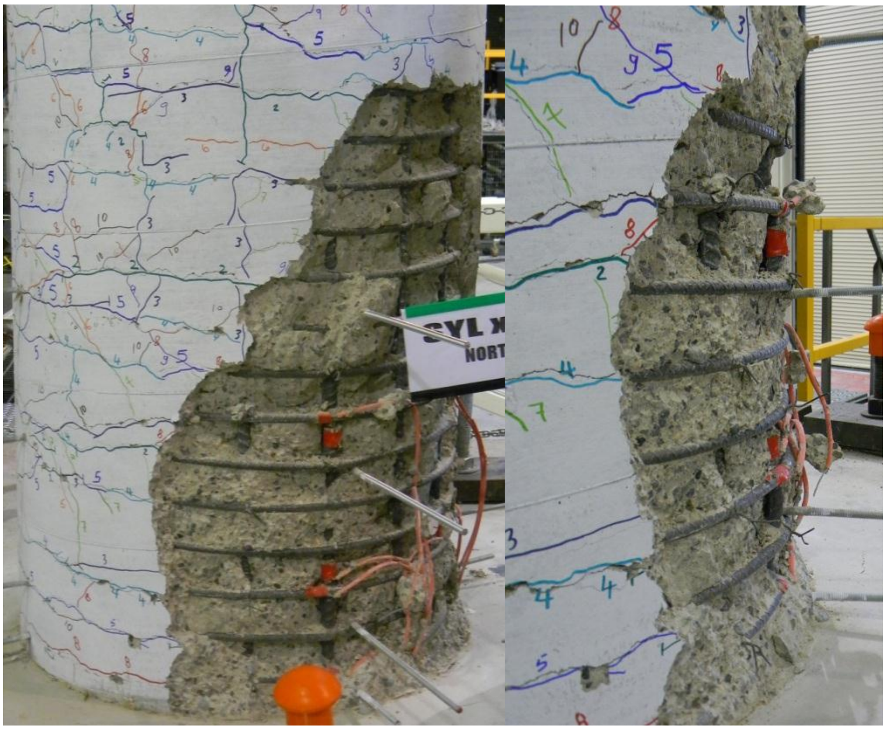

Figure 4.2: South Bent @ 350% of Design Earthquake

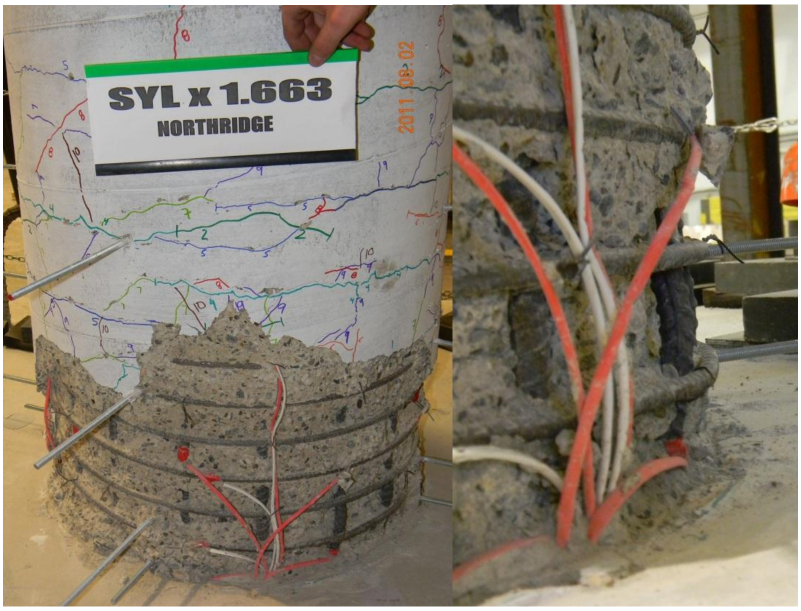

Figure 4.3: North Bent @ 350% of Design Earthquake

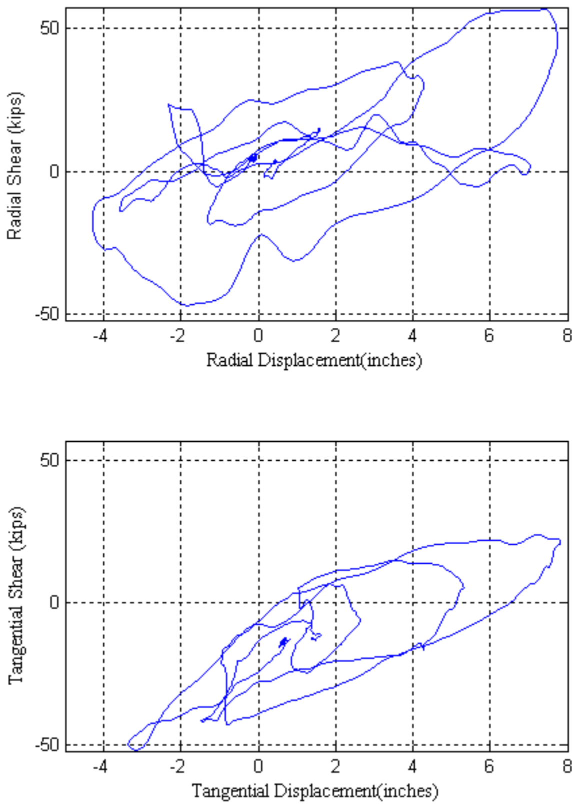

Figure 4.4: South Bent Shear vs. Displacement @ 350% Design Earthquake

During the preliminary analytical modeling of the system, it was noted that the girders were expected to uplift off of the slider plates approximately 1 inch (2.54cm) at 300% of the Design Earthquake. During the experiment, the 300% motion induced 0.6 inches (1.524cm) of uplift in-between the girders (for the location of vertical instrumentation, see Levi and Sanders, 2011, Figure 4.5. However, this displacement dramatically increased during the final ground motion, Figure 4.6. The displacement of the inner bay increased to approximately 2.5 inches (6.35cm). With this large increase in displacements, the experiment was concluded.

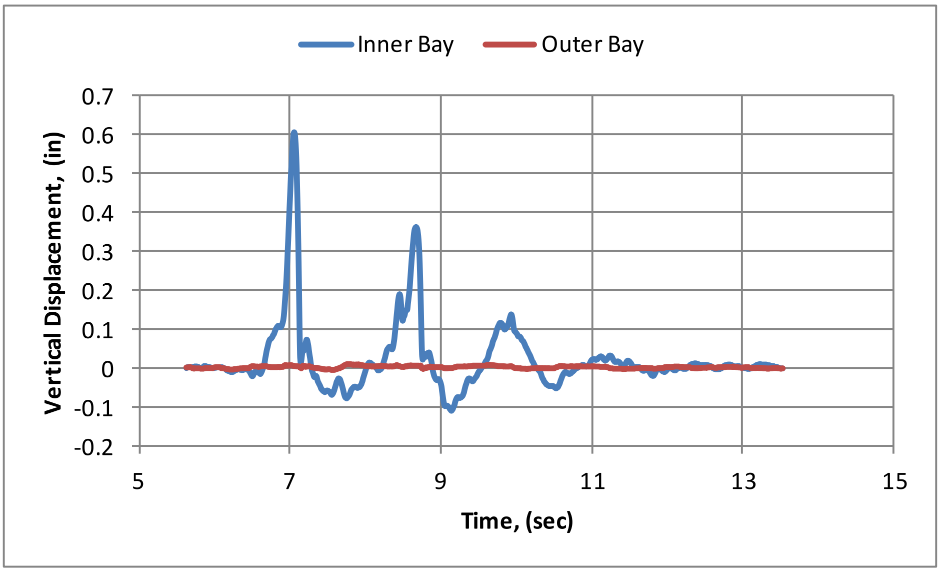

Figure 4.5: Vertical Uplift at North Abutment at 300% of the Design Earthquake

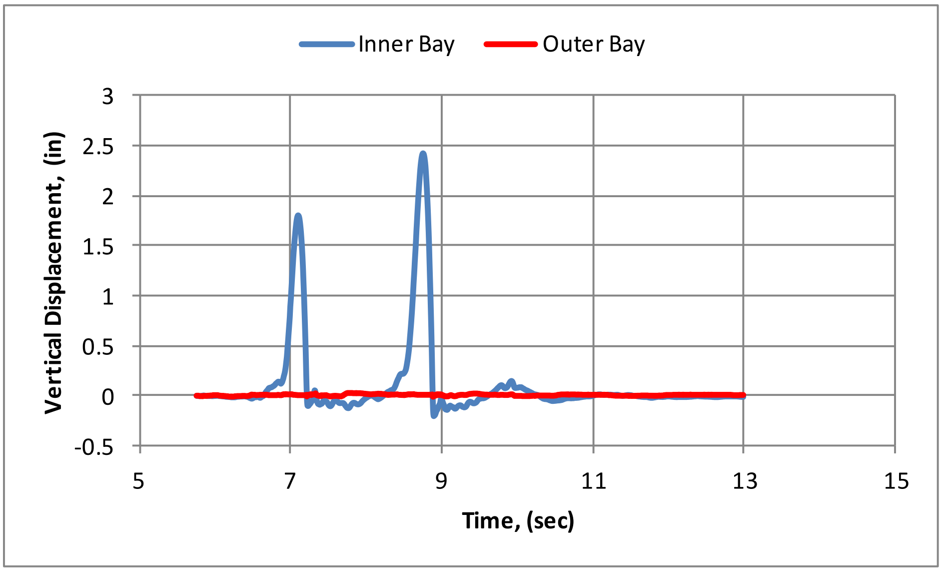

Figure 4.6: Vertical Uplift at North Abutment at 350% of the Design Earthquake

Torsional loading was expected from the radial restraint of the superstructure by the shear keys at lower level ground motions. In addition, the biaxial motion was expected to provide torsional loading after shear key failure. From the experimental results, the torsional loading was examined at 75%, the motion where the shear keys failed, and at 100%, to determine the effect of the shear keys in the system. Time histories of rotation about the vertical axis of each column are shown in Figures 4.7 and 4.8. From this data, it is clear that the torsional rotations in the system decrease after the loss of the shear keys. This is further shown in the plots of torsional loading in the columns given in Figure 4.9. Calculated from the load cells on the bent caps under each girder, the torque on the south bent decreases at the 100% ground motion. This reduction is due to the shear keys failing at the abutments allowing the superstructure to displace freely instead of being forced to move along the arc of a circle.
In the south bent, the torsional loading in the columns stayed at approximately the same level until 200% of the design earthquake. From this point forward, torsional loading in the system is caused by the excitation of torsional modes, and not boundary conditions. The data for the north bent in the figure is not shown for the 100 and 150% motions due to instrumentation malfunction.

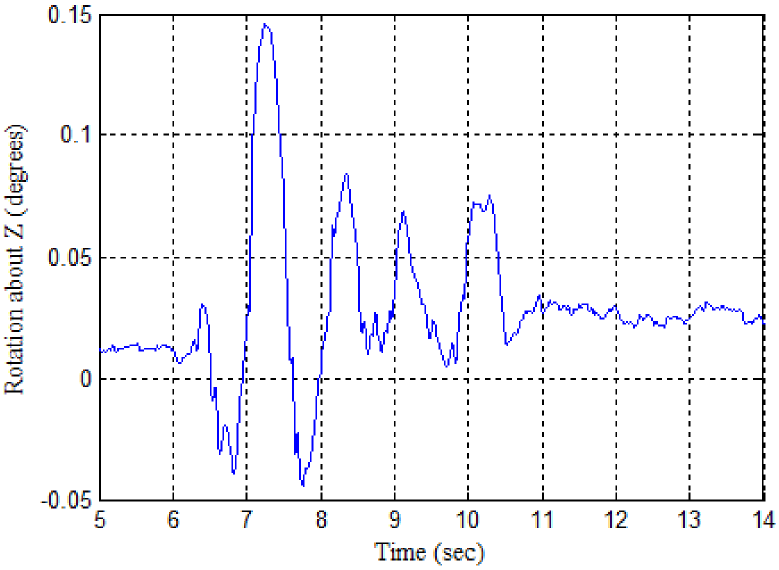

Figure 4.7: South Bent Rotation vs. Time @ 75% Design Earthquake

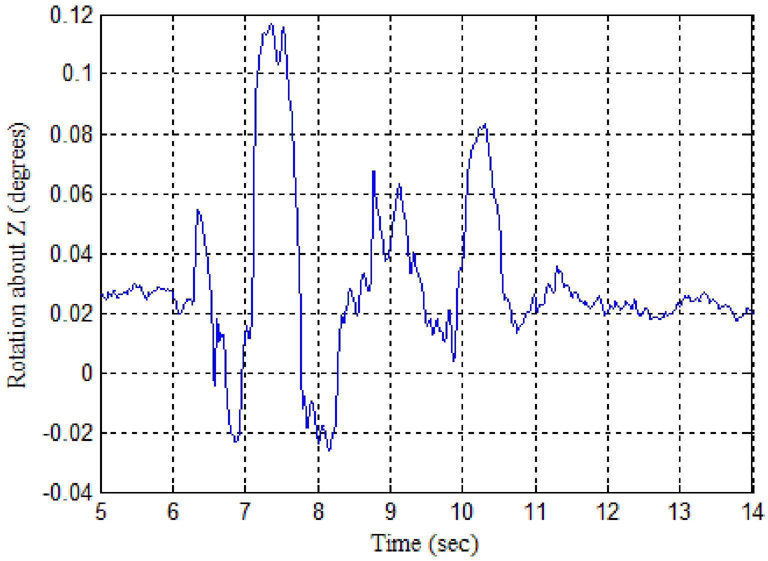

Figure 4.8: South Bent Rotation vs. Time @ 100% Design Earthquake

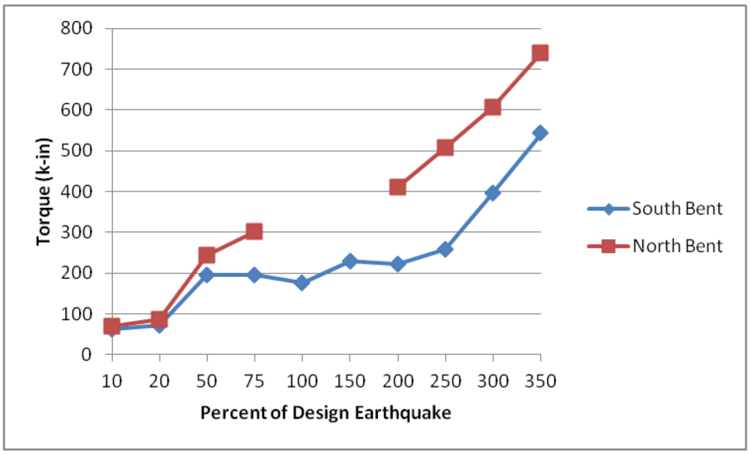

Figure 4.9: Column Torque

In addition to the previously mentioned items, curvature of the system was examined to determine if reverse bending, or double curvature, occurred in the system. Strain gauge data was investigated at the top of the column during testing. It was determined during testing that during the larger amplitude ground motions, yielding occurred in the longitudinal reinforcement at the top of the columns, thus, reverse bending should be considered for single column curved bridges.

5. ANALYTICAL COMPARISON

Preliminary comparisons of the analytical and experimental data have been made using SAP2000 (SAP 2000 v15.1.0). The analytical model of the bridge system uses a beam-plate system for the superstructure, such that the deck is modeled as shell elements, and the girders as frame elements. The top and bottom of the columns are modeled using fiber hinges to represent the plastic hinge regions. These models include the effects of strain penetration and strain rate loading effects, but they do not include P-Δ effects in the columns or friction at the abutments. Initial investigations show good correlation for the maximum displacement in the system, as seen in Figure 5.1. Further work is required to improve this match over the full time history of motion and to investigate reasons for remaining discrepancies.

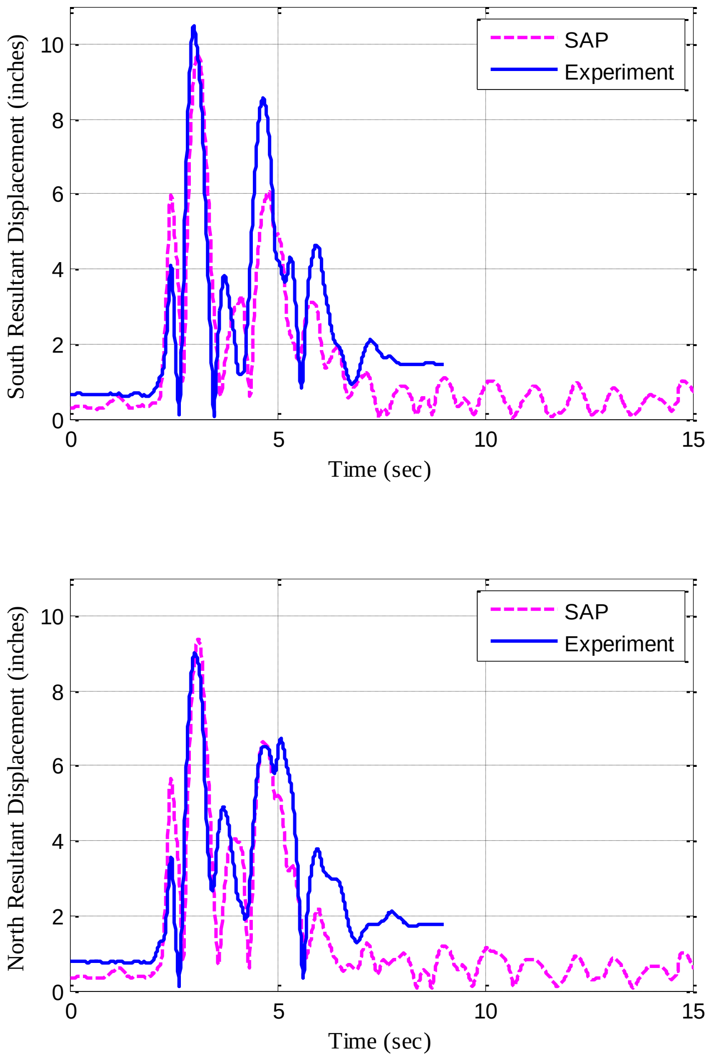

Figure 5.1: Resultant Displacement Comparison @ 350% Design Earthquake

6. CONCLUSIONS

The research being conducted at UNR has shown that the curvature of the bridge directly impacts the torsional loading and rotations at columns and bearings when shear keys are still intact. Once the shear keys fail, torsional loading and rotations change as system torsional modes are excited at higher ground amplitudes. In addition, uplift occurs at the abutments at large amplitude ground motions causing uplift of the middle and inner girders. Along with these discoveries, reverse bending of the columns is also expected and should be considered when designing single column bents in curved bridges. Experimental column investigation in curved bridge systems has provided information that could not be determined from component testing. The bridge performed very well with limited damaged until motions that were much larger than the design level earthquake.

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REFERENCES

AASHTO-LRFD Bridge Design Specifications, fourth Edition, 2007 and interims through 2008.

AASHTO Guide Specifications for LRFD Seismic Bridge Design, May 2007

BERGER/ABAM Engineers Inc., September 1996, "Seismic Design Course, Design Example No. 6."

Levi, M. and Sanders, D., "Seismic Response of Columns in Horizontally Curved Bridges," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-11-04, December 2011.

SAP 2000 v15.1.0, Computers and Structures, Inc.

XTRACT v3.0.5, Imbsen Software Systems.