SHAKE TABLE STUDIES OF SEISMIC PERFORMANCE OF A SEGMENTAL BRIDGE PIER

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ABSTRACT

Segmental bridge columns help expedite bridge construction and make the bridge available to the traveling public at a faster rate. The columns are often post-tensioned to provide continuity among the segments. Under strong earthquakes, post-tensioning has the added benefit of re-centering the columns and the bridge and reduce permanent lateral drifts. However, relying solely on post-tensioning to connect the column segments to each other and to the cap beam and the footings has a major drawback that the system has little energy dissipation capacity. There is also the added concern about damage at the junction of the segments, as the joints tend to open and close during earthquakes. A new pier model consisting of segmental columns was developed and tested as part of a 4-span bridge tested on three shake tables at the University of Nevada, Reno. To address energy dissipation issue, the connections between the end segments and the footing and pier cap provided full moment transfer with mild steel designed to yield. To limit the damage at the joints of the segments, unidirectional carbon fiber reinforced polymer (CFRP) fabrics were installed around the segments, which provided confinement for the entire cross section. The shake table testing consisted of small-, moderate-, large-, and very large-amplitude motions in two horizontal directions causing a maximum resultant drift ratio of exceeding 9 in exterior piers. The paper and presentation will describe a summary of the design, construction, testing, seismic performance, and analytical studies that were conducted after the tests.

Keywords: Segmental Pier; Acceleration Bridge Construction (ABC); Shake-table Testing; Seismic Design; Composite

1. INTRODUCTION

Accelerated bridge construction (ABC) is a potential contender to overcome the shortcomings of conventional bridge construction which is a time consuming process associated with traffic interruptions and risks to public safety. Prefabricated bridge systems are one of the key factors in achieving short construction period of ABC methods, and ease of construction and transportation to the site makes segmental columns a particularly attractive type. Past experimental studies using segmental construction of columns have focused on bridge components and shown promising results (Hewes and Priestly, 2002, and Kwan and Billington, 2003). Yamashita and Sanders (2006) performed shake table experiments and analytical studies to investigate the seismic performance of unbonded prestressed hollow segmental column and showed that using an unbonded prestress system provides excellent drift ratio capacity with limited permanent displacement. Motaref et al. (2011) showed that by providing monolithic connection between the end segments, footing and pier cap, the segmental column offers energy dissipation capability under seismic loading, which was the main drawback that prevents the use of standard segmental columns in high seismic hazard regions. Various combination of segmental construction with use of innovative materials have been tested in recent years (Yang and Okumus (2016), Nikoukalam and Sideris (2017)); however, knowledge of the behavior and performance of precast segmental bridge columns and their connections during an earthquake event in a bridge system is lacking.

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As part of a multi-university research program, National Science Foundation (NSF) funded a major research through the Network for Earthquake Engineering Simulation (NEES) to investigate the seismic performance of three quarter-scale 4-span bridge models with conventional and innovative materials using the multi shake table facilities of the University of Nevada, Reno (UNR). The first model was a conventional reinforced concrete bridge (Nelson et al. 2007), and the second model incorporated advanced material and details in plastic hinge zones (Cruz and Saiidi, 2010). The third bridge model incorporated various innovative ABC concepts such as using Glass fiber-reinforced polymer (FRP) tubes and Carbon FRP wrapping to provide confinement and shear resistance, and partial longitudinal resistance, segmental construction and precast columns with pocket connections, pipe-pin connection and implementing accelerated bridge construction techniques. The seismic performance of these details were investigated under bi-directional excitation (Kavianipour and Saiidi, 2013). The focus of this paper is to present the study of the third bridge model emphasizing on the behavior of the segmental pier in a bridge system and identifying its key performance data.

2. BRIDGE MODEL DESIGN AND CONSTRUCTION

2.1 Geometry

2.1.1 General Geometry

Since there were no prototypes available for bridges with composite piers and segmental column constructions, the geometry of the bridge model was selected based on 4-span prototype bridge with continuous post-tensioned reinforced concrete box girder superstructure supported on drop-cap two-column piers. The design of the conventional bridge model was based on the National Cooperative Highway Research Program (NCHRP) 12-49 Recommended LRFD Guidelines for the Seismic Design of Highway Bridges (ATC/MCEER 2001). The bridge model was a quarter-scale representation of the prototype, which was the largest possible scale to accommodate the limitation of the UNR shake tables such as payload, oil flow, and displacement capacity. The dimensions of the scaled bridge model were 32.6 m long and 2.3 m wide (Figure 1).

Each pier consisted of two circular columns with the same diameter and height, incorporating different details and construction methods. The exterior piers consisted of concrete-filled FRP tubes (CFFT), with
±55 degree glass fibers. These tubes served as formwork for casting the columns and provided flexural and shear reinforcement as well as confinement. Pier one, located on south shake table, had precast columns placed into a precast footing. Pier three had the same structural configuration but it was cast- in-place. These two piers were pin-connected to the superstructure using steel pipes. Pier two, which is the main focus of this paper, incorporated segmental reinforced concrete columns with post-tensioning rods and FRP wrapping with horizontal carbon fibers providing confinement and shear reinforcement. The typical sections of each pier is shown in Figure 2.

Figure 2. Pier typical sections

2.1.2 Exterior Piers (CFFT Piers)

For the exterior piers, CFFTs with glass fibers were used both as the formwork for the concrete, and hence reducing the construction cost, and for transverse and longitudinal reinforcement. The tubes were 370 mm diameter Red Thread® II pipes with wall thickness of 6.83 mm from NOV Fiber Glass System company production. The connection between columns and superstructure were provided by pipe-pins with 313.6 KN capacity, which used to eliminate the moments while transferring shear and axial force between columns and superstructure (Zaghi and Saiidi, 2010).

The difference between piers 1 and 3 was in the method of construction. Pier 3 was constructed using conventional cast-in-place methods while pier 1 was precast using pocket connections at the column-footing joints.

2.1.3 Interior Pier (Segmental Pier)
This pier consisted of two 4-segment columns, wrapped in two layers of carbon FRP (CFRP) sheets to satisfy shear and confinement reinforcement requirement. Motaref et al. (2011) suggests using steel ratio less than 1% to allow yielding of the bars prior to segment separation. For longitudinal reinforcement, eleven #3 bars with assuming yield stress of 413.7 MPa were used in each column, resulting in steel area of 780.64 mm$^2$ and longitudinal steel ratio of 1.0 %. The length of the top and bottoms segments was 609.6 mm and the length of the middle segments was 457.2 mm. Two layers of Carbon FRP with thickness of 0.1016 mm were used. The columns were connected to the superstructure by a 381 mm by 381 mm square pier cap. The top and bottom segments were casted monolithically with footing and pier cap, and consequently the fix connection at the top and bottom of the columns results in double-curvature behavior. The drop cap provided sufficient seat width for superstructure beams. The segments were connected to each other by a 34.9 mm unbounded high strength threaded rod that was prestressed with 266.9 kN force in central core. A 263 mm diameter PVC pipe was placed at the center of the footing, the segments, and the cap for passage of the unbounded post tensioning (PT) rod. The rod was anchored in the foundation below the columns and in the bent cap. Figure 3 presents the segmental pier layout and details.

### Figure 3. Segmental pier layout and details

#### 2.2 Bridge Model Construction

Superstructure beams and abutment structure, which consisted of an actuator buttress, a stack of four reinforced concrete block, and abutment seat support structure, were reused from previous studies (Nelson et al., 2007, and Cruz and Saiidi 2010). The affected areas from previous experiments were sufficiently retrofitted.

The piers were constructed outside the structures laboratory at the University of Nevada, Reno. For pier 3, first the formwork for the footing was built and then the bottom reinforcement layer was placed. The reinforcement cages for the columns were constructed and instrumented and then were hooked to the footing longitudinal bars. The GFRP tubes were centered such that the 25 mm concrete clear cover was maintained. The upper footing reinforcement mat was placed and tied, passing through pre-drilled holes.
in the GFRP tubes. Concrete was poured in two stages of footing pour and column pour.

For pier 1, the precast pier, the procedure was very similar. The reinforcement cage of the footing was placed in the formwork and the concrete was cast. The precast column reinforcement cage was set up and instrumented and the concrete was poured. After the concrete set, columns were moved by a forklift and placed into octagonal holes in the footing, and the gap was filled with high strength grout.

The segmental pier construction was different. The formwork and reinforcement cage for the footing and pier cap were built separately. The reinforcement cage for all four segments of the columns were constructed and instrumented. Reinforcement cages for top and bottom segments were tied to the footing and pier cap, respectively. The PVC pipe was placed at the center for passing the high strength tendon in order to post-tension the segments together. Concrete cast for the segmental pier consisted of three stages: 1- casting footing and pier cap concrete, 2- casting base and top segments, and 3- match-casting second and third segments on base and top segment, respectively. A layer of chemical liquid concrete bond breaker was applied on the surface of lower segment to facilitate the removal of the upper segment. After the concrete was cured, the surface of each segment was ground, cleaned, and covered with a thin layer of epoxy and then each segment was wrapped by two layers of CFRP (Figure 4). The curing time for FRP was seven days. The high strength tendon was instrumented and passed through footing, pier cap, and the four segments of each column. Before assembling, the PT rod was tied at the bottom of the footing by a nut and a steel plate. Interface surfaces between segments were glued by small amount epoxy to stabilize the segments during construction. Rectangular holes were built in pier cap and footing of the segmental pier for inserting anchor plates and bolts for the post tensioning rods. Figure 5 shows the assembled segmental bent that used four pieces of steel members along the columns and tightened with straps to keep the columns straight. The last step of the assembly was post-tensioning the PT rod with a 266.9 kN force.
Different components of the bridge were transported to the lab and they were assembled according to construction drawings. To avoid slippage between the elements, a layer of hydrostone was used between adjacent parts. Longitudinal and transverse post-tensioning was implemented in superstructure to provide a rigid diaphragm. To measure the dynamic response of the bridge model and monitor various aspects of the behavior, a large number of strain gauges, Novotechnik displacement transducers, load cells, and accelerometers resulting in total of 296 data acquisition channels were installed on the bridge model. Moreover, 52 channels recorded data such as displacement, velocity, acceleration, and oil pressure from shake tables and actuators. Figure 6 shows the assembled bridge model in UNR structures lab.
3. SHAKE TABLE TESTING

3.1 Shake Table Input Motions

The response of the bridge model is greatly dependent on the amplitudes and frequencies of the input motions and their sequence. Thus, preliminary studies using the Open System for Earthquake Engineering Simulation (OpenSees) program (OpenSees, 2002) was performed to predict the performance of the bridge model. As shown in Table 1, the applied input motions was a series of modified 1994 Northridge, California earthquake record (Century city station) with increasing amplitudes (Kavianipour and Saidi, 2013).

![Table 1. Shake table testing applied loading protocol.](image)

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3.2 Test Results

To understand the bridge model performance, both the observed damage progression and measured test results were closely monitored.

3.2.1 Observed Results

Damage tracking process during the experiment consisted of physical inspection that documented with both written notes and pictures, and video recordings from strategic points during the tests.

The progression of damage for before test, run 5 and after test for top segment is shown in Figure 7. Minor cracking existed on the pier cap before the start of the experiment. During runs 1 through 3, no perceptible damage occurred to segmental pier, as the initial cracks did not increase. Small cracking in the footing started after run 4 and progressed slowly in further runs. During run 6, minor shear cracking was initiated in the pier cap which progressed rapidly to spalling in run 8 and considerable rupture of the concrete in run 9 (Figure 8). The segmental columns showed no sign of damage. The CFRP wrapping remained intact and without any rupture. Due to the small moments at the segment joints, there was no separation between column segments, and no bleeding of the concrete was observed. This was different from the results showed by Motaref et al. (2010), in which substantial opening was observed between the segments due to higher moment demands at segment interface. After the test, CFRP wrapping was removed and the only cracks on the concrete surface were in the place of nominal hoops.
Figure 7. Progression of damage before the test, after run 5, and after the test in top segment

Figure 8. Large crack in the pier cap of segmental bent

3.2.2 Measured Results

Post-processing methods such as unit transformation and filtering were performed on the raw data from data acquisition portal to investigate the primary response indicators of the earthquake excitation. Figure 9 presents the segmental pier force-displacement curve in the transverse direction. Segmental pier performed relatively elastic for the first three runs, and it exhibited nonlinear behavior and widening of loops in subsequent runs, indicating increase hysteretic energy dissipation. This energy dissipation took place mostly through the yielding of the longitudinal bars in the base and top segments. The PT rod remained elastic during the experiment. Compared to the exterior piers, segmental pier attracted the highest lateral force and the lowest displacement due to higher stiffness.
The maximum transverse and resultant drift ratio for segmental pier occurred during the high amplitude runs (runs 9 and 10), with values of 5.46% and 7.15%, respectively. In runs 1 through 8, the relative transverse displacement between pier cap and superstructure were less than 3 mm and negligible compared to the total relative displacement of the bent. However, this displacement reached a value of 66.15 mm in run 9, and this slippage was due to the loosening of the post-tensioning rods that connected the superstructure beams to pier cap. The residual relative displacement after the test was 28.83 mm. Also, the maximum gap opening during the experiment occurred in run 9 between the base and second segment with a value of 12.2 mm. This interface was the most critical surface since it was near the bottom plastic hinge area.

4. ANALYSIS OF EXPERIMENT RESULTS

To understand the performance of segmental pier under seismic excitation, various factors were investigated by analyzing the measured data. Study of average strain in longitudinal steel reinforcement showed that strains at the bottom plastic hinge zone were on average 130% higher than those of top plastic hinge zone, indicating that the flexural demand at the base was higher due to the flexibility of the pier cap. Figure 10 compares the maximum strains in the base and top segments in each run.

The accumulated energy dissipation in transverse direction for all three piers in runs 3 and 10 are shown in Figure 11. With run 3 being the onset of yielding in the longitudinal bars at the footing-column interface, more seismic energy was dissipated through yielding of steel in top and bottom segments through each subsequent run.
The maximum recorded axial force in the PT rod was 94% of the 978.6 kN yield strength and exhibited the re-centering capability that reduced the residual displacement. The magnitude of the residual displacements was insignificant. The largest resultant residual displacement of 24.7 mm, equal to 1.06% drift ratio, for segmental pier occurred in the experiment, while reaching a maximum of 7.15% resultant drift ratio. The segmental pier residual drift ratio is significantly smaller than those for piers with reinforced concrete and innovative materials (Nelson et al., 2007, Cruz and Saiidi, 2010). Figure 12 illustrates the progression of resultant residual drift ratio during the experiment.

5. CONCLUSIONS

Segmental piers can serve as one of the more effective solutions to reducing construction time and roadway interruptions as part of ABC methods. The seismic response of a segmental pier as a part of a bridge system was investigated as a part of this study and the results showed that the details for segmental construction can be used in regions with high seismicity due to the ability to sufficiently dissipate seismic energy and performing as ductile members. Using integrated base and top column with footing and pier cap engages the longitudinal steels in energy dissipation process through yielding and the post-tensioning rod serves both as the connecting elements between segments and re-centering component to reduce permanent deformation. The damage in the pier cap is easily repairable without the complications or required rehabilitation time that would be expected for damaged columns after the earthquake.

6. ACKNOWLEDGMENTS

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