SHAKE TABLE EXPERIMENTS OF PRECAST, PRETENSIONED BRIDGE

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Abstract

This paper describes the verification by shake-table testing of a bridge bent system that was designed to be rapidly constructible, and to provide superior seismic performance through re-centering and reduction of damage. The system used precast concrete elements, and the re-centering was achieved by means of unbonded pre-tensioning in the columns. Column damage was suppressed by steel shoe detail that confined the ends of the columns. A two-span, three-bent bridge was tested seismically on the shake tables at the NEES Facility at the University of Nevada, Reno. The bridge was quarter scale, had two-column bents with 12" diameter columns, and 30-ft. span lengths. The bridge geometry was similar to that of one previously tested at the University of Nevada, Reno that used conventional non-prestressed, cast-in-place concrete columns.

Introduction

Within the United States, design of reinforced concrete bridges in seismic regions has changed little since the mid-1970s, when ductile details were first introduced. Many bridge bents in seismic regions are constructed of cast-in-place reinforced concrete. Cast-in-place bridges with proper confinement have performed well in the past, but to meet modern design expectations for bridges, new structural systems and construction methods are needed to improve: 1) speed of construction, 2) seismic resilience and 3) durability.

The new system was originally developed at the University of Washington has the following key features: 1) columns and beams are cast off-site and then assembled rapidly once they arrive on site, 2) construction is further accelerated by using a "wet socket" connection between the column and the spread footing (Haraldsson et al 2012) and a "hybrid-bar-socket" connection between the column and the precast beams (Davis et al 2011), 3) post-earthquake residual displacements are reduced by pre-tensioning the precast bridge columns with unbonded tendons, which are designed to return the system to its original position when the ground motion stops, and 4) damage to the system is minimized by incorporating a confined rocking detail, or "shoe", at the column ends.

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Bridge Specimen

The shake-table specimen was designed to investigate the global response of the pre-tensioned, rocking bent system. The bridge geometry, illustrated in Figs. 1 and 2, was chosen to match that a specimen previously tested at University of Nevada, Reno (Johnson et al 2006) that used conventional, non-prestressed, cast-in-place concrete columns. The bridge specimen was quarter scale with octagon columns ended by steel shoes. Bent dimensions and column reinforcement details of the shake-table specimen are shown in Fig. 3 and 4. The bridge length was 69.25 ft. (21.11m); The clear height of the specimen was 127 in. (3.21 m); the total imposed weight on the bridge was 170.2 kips (757.4 kN).

Fig. 1. Shake table specimen

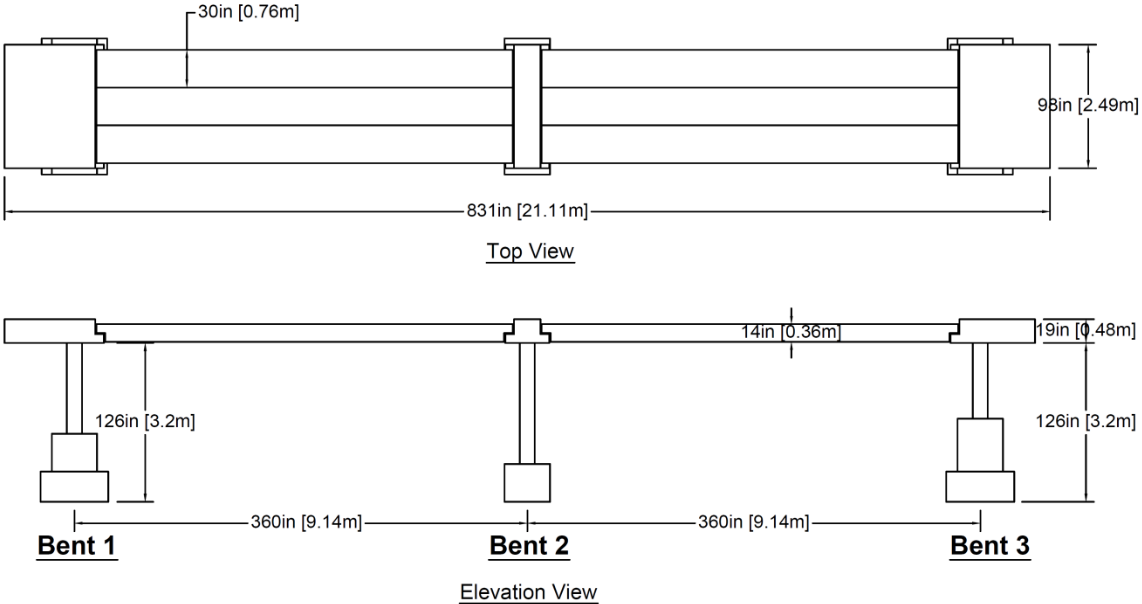

Fig. 2. Overall dimension of the shake-table specimen.

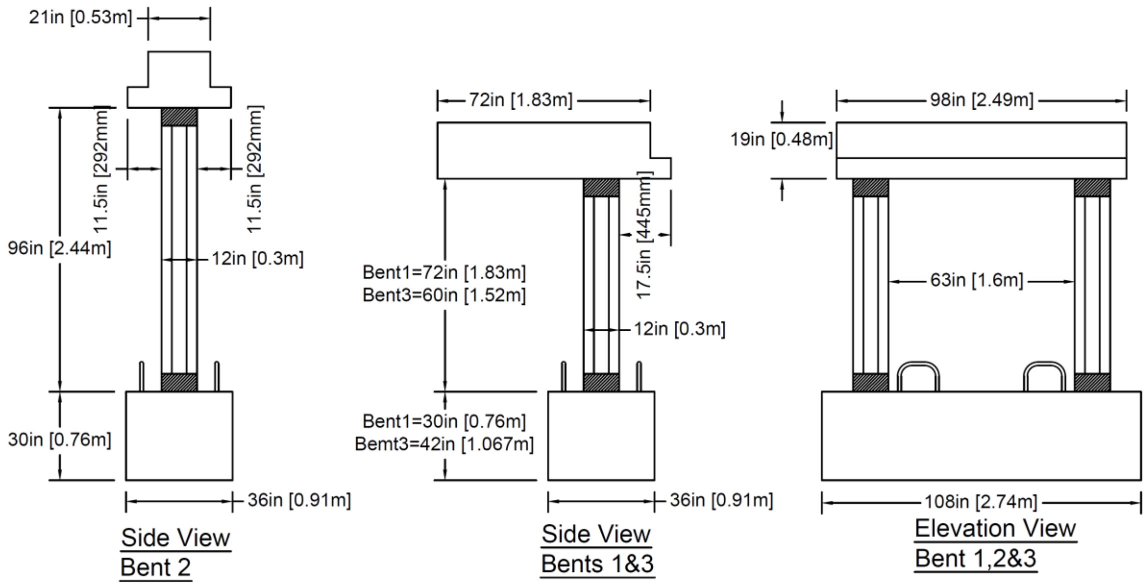

Fig.3. Bent dimensions.

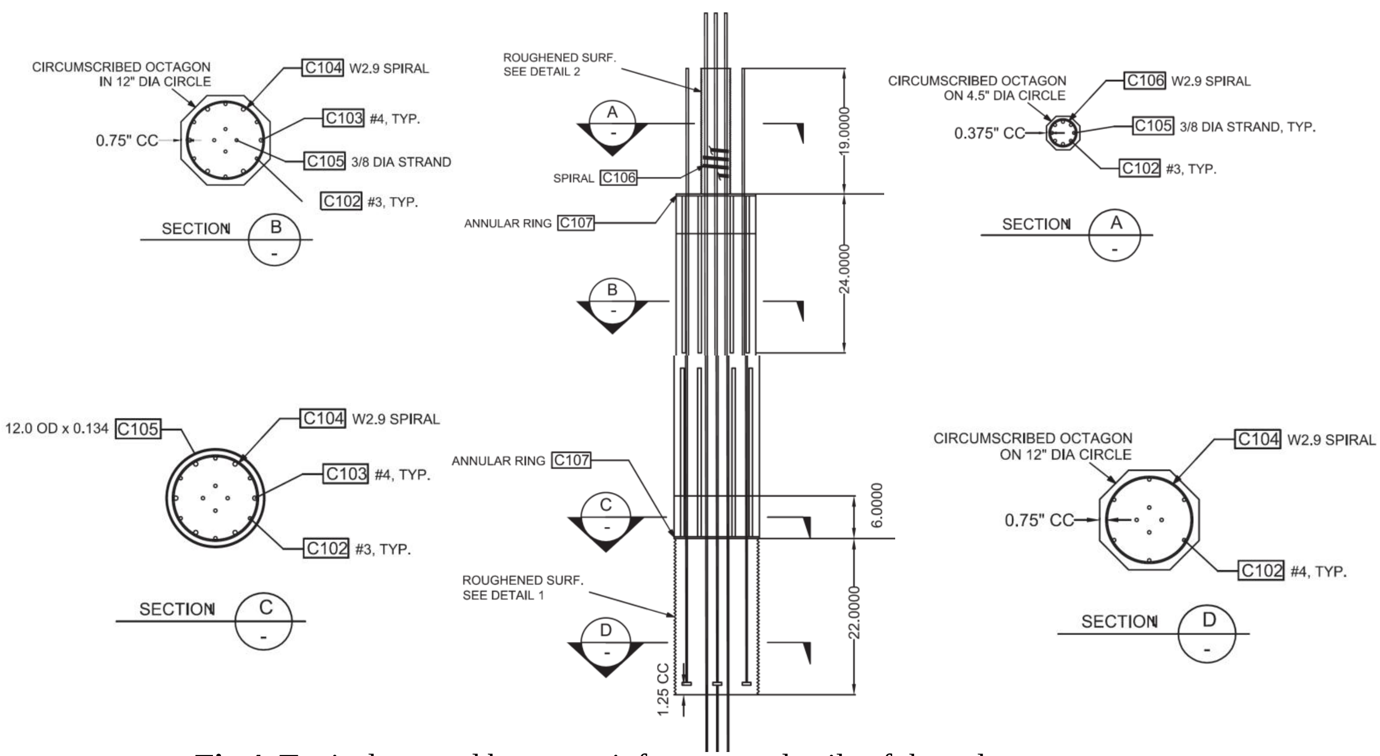

Fig.4. Typical top and bottom reinforcement details of the columns.

Specimen Design

Column Design

The column clear heights, from the top of the footing to the bottom of the bent caps, varied. This matched the column heights of the previous bridge experiment tested that used conventional non-prestressed columns. Clear heights of 6 ft. (1.83 m), 8 ft. (2.44 m), and 5ft (1.52 m) were used for bents 1, 2 and 3 respectively. The column bases were embedded 22 in. (0.56 m) inside the cast-in-place combined footing using a wet socket connection (Haraldsson et al 2012). The tops of the column were integrally grouted into the superstructure using a hybrid-bar-socket connection (Davis et al 2012). The column longitudinal reinforcement consisted of 6-#3 bars and 4-3/8 in. diameter epoxy coated prestressing strands. The longitudinal reinforcement was debonded at each rocking interface over sufficient length to prevent bar failure at designed deformation. The strands were bonded in the footing and bent cap and unbonded through the column clear height. The concrete at the column-to-footing and the column-to-cap-beam connections were confined by a steel rocking detail, which consisted of a circular steel pipe welded to an annular end plate. The end plate was intended to concentrate column rotations at the two interfaces, creating nearly rigid-body rotation of the columns in between. Supplementary reinforcing was welded to the end plate, extending into the clear height of the column to distribute compressive forces and arrest cracks that could form at the boundary of the steel confining tube.

Superstructure Design

The superstructure of the bridge consisted of six precast slabs post-tensioned together to provide a stiff deck. The slabs had been designed for the conventional bridge and were reused in this bridge. Each span, consisting of three slabs, was assembled on the lab floor and post-tensioned transversely using ten 1.25-in. diameter rods. Each rod was prestressed to 100 kips (445 kN) of force, to rigidly attach the precast slabs together, to prevent slippage and to provide flexural capacity in the transverse direction. Second each set of beams was placed between the bents and aligned with longitudinal post-tension ducts embedded into the three precast caps. The deck was longitudinally post-tensioned to a total force of 720 kips (3204 kN) to provide rigid connections between the bent caps and slabs.

Specimen Construction

The key stages of bridge construction are shown in Fig. 5. Six columns and three bent caps were cast at University of Washington and shipped to University of Nevada, Reno. The columns were aligned in the footing formwork, and the spread footings were cast in place in an outdoor staging area. The footings and columns were then moved as a single piece onto the shake tables. Due to the variations in column heights, spacer blocks were used between the bottom of the footings and the shake tables to maintain a level superstructure. To connect the cap beams to the columns, two types of grout were used. A fiber reinforced grout pad was used at the column-to-cap-beam interface to allow for the adjustment of the cap elevation and level. A standard, non-fiber reinforced, grout was used to connect the column's reduced section and longitudinal reinforcing to the bent cap. The placement of the non-fiber grout was postponed until after the longitudinal post-tensioning was concluded to reduce secondary moments in the columns due to slab shortening.

After the bent and spacer blocks were aligned, grouted and vertically bolted to the shake tables the preassembled spans were supported on formwork between each bent cap. Hydrostone was placed between the bent caps and the slabs, and the spans were lowered onto the bent cap ledges. The post-tensioning was conducted in stages to allow the placement of the superimposed mass. The procedure was designed to minimize the secondary moments on the columns due to post-tensioning. Eight concrete blocks with a total weight 160 kips (712 kN) and 10.2 kips (45.4 kN) of steel plates were placed on the superstructure to provide a representative structural mass, scaled from the superstructure of the prototype bridge.

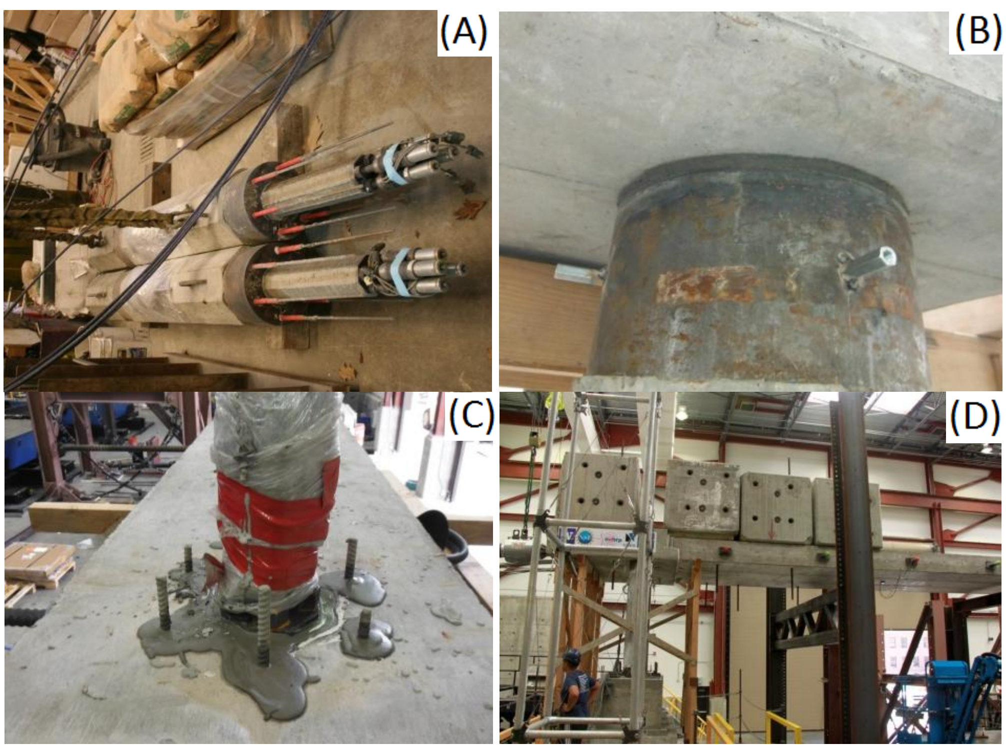

Fig.5. Photographs of the construction phases of shake table specimen: (a) precast columns at University of Washington; (b) fiber grout between column and bent cap; (c) Non-fiber grout between column reduced section and bent cap; (d) superimposed masses.

Instrumentation

The bridge was instrumented with 395 channels to record accelerations, displacements, bar/strand strains, and changes in the strand forces using load cells. A summary of the instrumentation plan is shown in Table 1. Transverse, longitudinal, and vertical accelerations of the superstructure at each bent and midspans were measured using accelerometers. Superstructure displacement and column curvatures were measured using displacement transducers. The strains in the longitudinal reinforcement, transverse reinforcement, longitudinal strands and steel shoe within critical column sections were measured with strain gauges. Potential slippage of strands at top of the columns was measured using load cells.

Table 1. Instrumentation Summary

Recorded response	Count
Potentiometers	
Slab displacements (T, L, V)	25
Column curvatures	72
Table displacements (T, L)	6
Accelerometers	
Slab accelerations (T, L, V)	15
Table accelerations (T, L)	6
Table velocities (T, L)	6
Strain gauges	
Longitudinal reinforcement strain	165
Transverse reinforcement strain	24
Strand strain	41
Steel shoe (Rosette)	6
Load cell	
Strand load cell	23
Actuator load cell	6

Test Schedule

Both low- and high-amplitude earthquake excitations were used to investigate the bridge response; a summary of the test schedule is shown in Table 2. The excitations were based on the 90 deg. and 360 deg. components of the Century City Country Club North (CCN90/CCN360) record from the 1994 Northridge California Earthquake; the 360 deg. component of the Sylmar- Olive View Med. Center (SYL360) record from the 1994 Northridge California Earthquake; and the 0 deg. component of the Takatori (TAK000) record from the 1995 Kobe, Japan Earthquake.

Low-amplitude motions consisted of coherent, incoherent and biaxial motions, whereas high-amplitude motions consisted of only coherent motions in the transverse direction of the bridge due to the absence of abutments. White-noise and square wave excitations were distributed throughout testing to track the bridge properties including the bridge periods and damping. Sinusoidal waves were added to evaluate the dynamic response of the bridge subjected to harmonic motions. Because of the one-quarter geometric scale, the time coordinate of the input was multiplied by a factor of 0.5.

Since the main objective of this study was to compare the response of the precast, pre-tensioned bridge bent system with the conventional cast-in-place bridge previously tested at the University of Nevada, Reno, a majority of the motions used were the same as in the previous experiment. Preliminary OpenSees models were used to evaluate the effects of adding Sylmar, Takatori motions and sinusoidal motions to the test schedule. The intent was to add these motions without altering the system performance during later motions that were comparable to the previous experiment. The final motion schedule eliminated some low-amplitude motions from the previous experiment and added Sylmar and Takatori motions at high-amplitude motions.

To investigate the bridge behavior with different excitations including near fault motions, Sylmar and Takatori motions were added after motion 14. The acceleration histories were scaled to have similar structural demands to the Century City motion.

Observed Damage

During the low-amplitude motions, no damage was observed in the columns or the superstructure. Similarly, during the high-amplitude motions, no damage (Concrete cracking, slippage of the longitudinal post-tensioning and cracking the non- fiber grout) was observed in the superstructure.

The first yield of the longitudinal reinforcement occurred during Motion 13. The first rebar fracture occurred at bent 1 during Motion 17, at a maximum column drift ratio of 5.7%. Flaking of the column concrete first occurred above the steel confining tube during Motion 16. Bulging of the steel shoe occurred in Bent 1 and Bent 3 during Motion 18. Multiple rebar fractures and grout pad loss occurred during Motion 18 after exceeding drift ratios of 9% and 6% for bents 1 and 3 respectively, and during Motion19, after exceeding drift ratios of 11% and 13% for bents 1 and 3 respectively. Fig. 6 shows the damage progression of the column concrete and steel shoe for Bent 1 at the end of Motion 19.

Table 2. Test Schedule with motion description.

Test	Test Type	Description	Test	Test Type	Description
1A	Low Level Coherent Motion	CCN90 (0.08g PGA)	S4	Sinusoidal Motion	0.15g 0.30sec
1B		CCN90 (0.15g PGA)	S5		0.10g 0.30sec
4	Low Level incoherent Motion	CCN90 (0.07g-0.18g-0.18g)	14B1	High Level coherent Motion	SYL360 (0.20g PGA)
5		CCN90 (0.18g -0.07g-0.18g)	14B2		SYL360 (0.40g PGA)
6		CCN90 (0.18g -0.18g-0.07g)	14C		TAK000 (0.20g PGA)
9A	Biaxial Motion	CCN90/CCN360 (0.08g PGA)	15		CCN90 (0.5g PGA)
9B		CCN90/CCN360 (0.15g PGA)	16		CCN90 (0.75g PGA)
12	High Level coherent Motion	CCN90 (0.08g PGA)	17		CCN90 (1.00g PGA)
13		CCN90 (0.15g PGA)	18		CCN90 (1.33 g PGA)
14A		CCN90 (0.25g PGA)	19		CCN90 (1.66 g PGA)
S1	Sinusoidal Motion	0.05g 0.25sec	20A		CCN90 (0.75g PGA)
S2		0.10g 0.25sec	20B		SYL360 (0.843g PGA)
S3		0.15g 0.25sec	21A		TAK000 (0.40g PGA)
9C	Biaxial Motion	CCN90/CCN360 (0.25g PGA)	21B		TAK000 (0.611g PGA)
			21C		TAK000 (0.80g PGA)

Measured Results

The bridge induced low displacement/drift levels during the low-amplitude motions; while during the high-amplitude the displacement/drift level was high (maximum drift was 13.2% for Bent 3). The maximum residual drift was 0.2% for Bent 3 during motion 19. The bridge was subjected to three design-level earthquakes after the highest motion. Motion 19 was equivalent to 2.2 times the design earthquake. The bridge showed superior resistance for these motions with maximum residual drift equal to 0.1%.

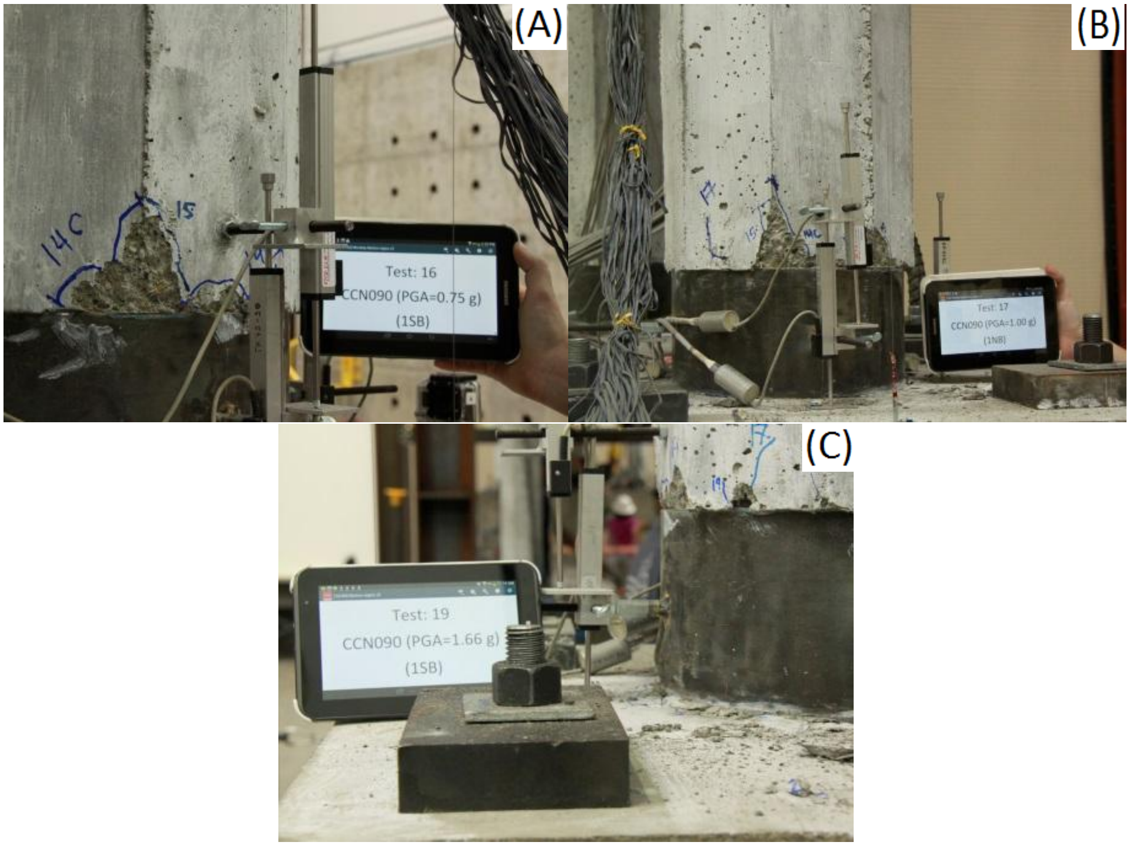

Fig.6. Damage progression for bent 1 at bottom of each column: (a) flaking at bottom of south column of bent 1 at motion 16; (b) flaking at bottom of north column of bent 1 at motion 17; (c) shoe bulge at bottom of south column of bent 1 at motion 19.

<u>Comparison with Conventional Bridge</u>

When compared to the bridge with conventional non-prestressed columns, the results show that the new system produced the same displacement/drift ratios up to motion 16. Starting from Motion 17, the new system produced higher displacements than the conventional bridge. The new system had a maximum drift of 13.2% for bent 3, while the previous bridge had 8.0%. The residual displacements for the new system were much lower than the conventional system. The new system had 0.2% for Bent 3 for Motion 19, while the pervious bridge had 0.5%. Fig.8 and Fig.9 show the comparison between the new design and the conventional design. For both maximum drift ratio and residual drift ratio for Bent 1. The researchers are currently evaluating other possible reasons for differences in the response; reasons could include difference in the shake table response, changes in bridge period, and/or the differences in the cyclic, force-deformation characteristics of the bridges.

The new system showed less damage than the conventional bridge. Fig 7 shows the comparison between the damage at end of Motion 19 for both specimens. The new system experienced minimal spalling and rebar fracture, wereas the conventional bridge sustained total failure of bent 3 including excessive spalling, spiral fractures and bar buckling. After Motion 19, equivalent to 2.2 times the design earthquake, the specimen continued to resist lateral forces and showed excellent re-centering. The maximum residual drift ratio for these motions was less than 0.3%. This is in contrast to the previous experiment where, after Motion 19, the superimposed mass was removed from bent 3 due to concerns of collapse, allowing the test to continue.

Fig.7. Damage comparison between the new bridge system and conventional bridge after test 19. (Right figure from test 19 of conventional bridge (Johnson et al., 2006)

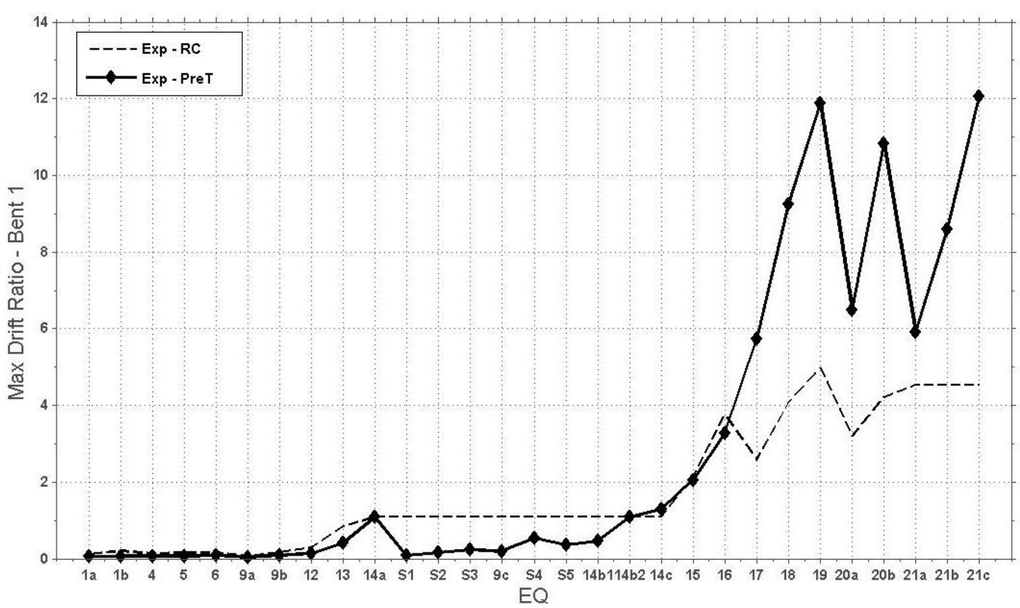

Fig.8. Maximum Drift Ratio comparison between the new system and conventional bridge for bent 1.

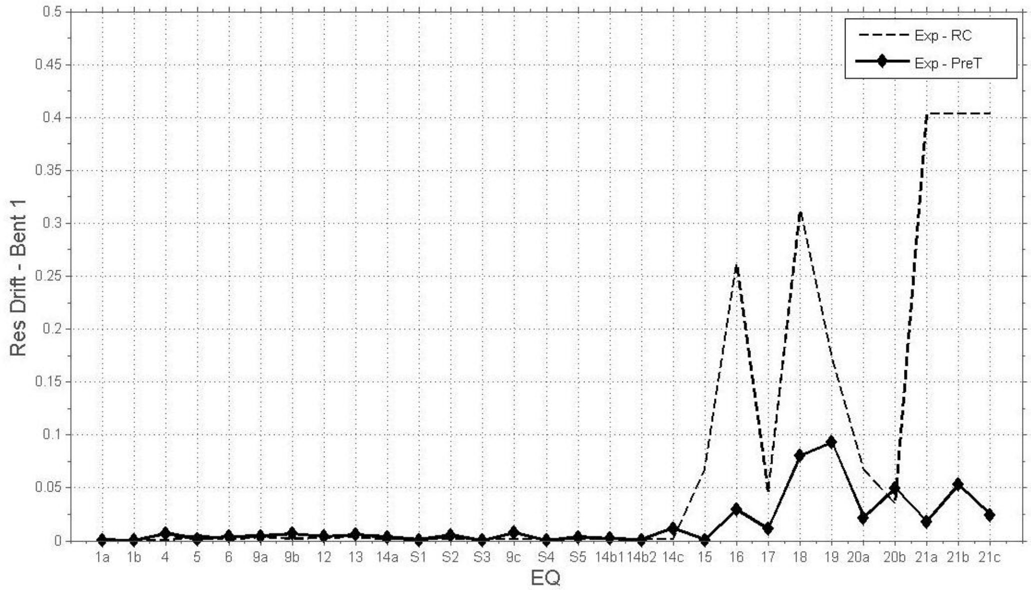

Fig.9. Residual Drift Ratio comparison between the new system and conventional bridge for bent 1

*Note: Since motions were added to the conventional loading protocol for the new system, straight lines are used for the results of conventional bridge

Conclusions

A new bridge system has been developed for use in any seismic region. It accelerates bridge construction, it re-centers after extreme earthquakes, and it minimizes seismic damage.

1. Damage is minimized by rocking, confinement details.
2. Re-centering is achieved by pre-tensioned strands
3. Compared to the conventional bridge, the new bridge induced less observed damage with no exposure of column reinforcement occurring during any test.
4. Compared to a conventional bridge, the peak transverse displacements for the new bridge system were higher on average when subjected to a high-amplitude ground motion, while the new bridge system demonstrated lower residual displacements for all experiments.

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