FULL SCALE TESTING OF CONCRETE BEAM-COLUMN JOINTS USING ADVANCED CARBON-FIBER COMPOSITES

Kevin C WOMACK[1], Marvin W HALLING[2] And Robert M MOYLE[3]

SUMMARY

The cost of retrofitting or replacing deteriorated bridge structures is astronomical. Although seismic upgrading is possible through traditional methods of repair, the rapid decay of concrete and reinforcing steel caused by the freeze/thaw cycle and road salts has forced the Utah Department of Transportation (UDOT) to look at alternative methods and materials for repairing corroded and deficient structures. One of these alternative methods is the placement of advanced carbon-fiber composite wraps around the areas of spalled concrete and corroded rebar. This composite wrap may not only aid in the prevention of natural and chemical deterioration, but may also add to the seismic capacity of structures.

Research at the University of California at San Diego (Seible, Hegemier, Priestley, Innamorato, Weeks, and Policelli 1994) has shown that non-ductile concrete columns retrofitted with various composite wraps have exhibited increases in ductility. However, there remains a need to investigate the performance of the application of advanced carbon-fiber composites to typical reinforced concrete column/bent-cap joints. This study addresses the issue of retrofitting reinforced concrete column/cap joints with advanced carbon-fiber composites using full-scale laboratory specimens.

UDOT, in conjunction with researchers from Utah State University (design and execution of full scale experimental work), the University of Utah (analysis and design of composite wraps; Gergely, Pantelides, Nuismer and Reaveley 1998), and the XXsys Corporation of San Diego (application of composite wraps to experimental specimens) began in late 1995 to investigate the application of these advanced carbon-fiber composites to a bridge pier cap and column joint. The Interstate 80 overpass crossing Highland Drive in Salt Lake City, Utah was chosen as a model for this investigation. The pier to be retrofitted with graphite composite, with identical beam-column joints to be tested in the lab, is a three column pier as shown in Figure 1.

This report outlines the work done by the research team at Utah State University in designing, fabricating, testing, and analyzing data from six full-scale test specimens to validate the ultimate capacities and ductilities of the carbon-fiber retrofitted beam-column joints.

INTRODUCTION

The physical testing of six full-scale models was executed at Utah State University. Two of these tests were performed on bare reinforced concrete column/bent-cap joints to establish the as-is condition of the existing bridge. Advanced carbon-fiber composites in various configurations were applied to the remaining four models in order to test the effectiveness of various retrofit designs using this material. Of particular interest were changes in joint stiffness, strength, and ductility, as well as the stress characteristics of the composite material. The full-scale modeling phase of the project was broken down into three major components: design and construction; instrumentation and testing; analysis and results.

[1] *Department of Civil Engineering, Utah State University, Logan, Utah 84322-4110*
[2] *Department of Civil Engineering, Utah State University, Logan, Utah 84322-4110*
[3] *ARW Engineers, Inc., Ogden, Utah*

FULL-SCALE MODELING

Design and Construction

After several design ideas were considered, a self-contained configuration was chosen as the design for the test specimens as shown in Figure 2. Note that this configuration is actually an inverted replica of the existing bridge pier without the center column. The specimens were placed on three supports as shown. Two supports were placed directly under the columns and another under the center of the beam at the theoretical point of inflection for a deflected bridge pier subject to a lateral load. These supports were built 381mm (15 inches) high in order to accommodate flexure of the beam under loading and to allow enough room for workers to place composite wraps under the specimens.

In order to simulate the deterioration of the stirrups in the bent-cap and column of the existing structure, four stirrups in the haunch just outside the column, three stirrups in the beam just inside the column, and the first three stirrups on the column next to the beam were cut at the centerline of the beam and the column. Cutting the stirrups at the centerline of the elements, which is close to the neutral axis for bending (where bending stresses are zero and shear stresses are a maximum), created a maximum impact on shear resistance and confinement. This is where the deteriorated stirrups would create significant problems.

The concrete for the models was placed in two separate sequences. This created a cold joint at the column and beam interface similar to the one on the existing I-80 structure. The concrete used in the model was a UDOT Class-AA mix. This calls for a course aggregate size from 25.4 mm (1 inch) to number 4 sieve, a minimum cement content of 6.5 sacks per cubic yard, a total slump of 25.4 to 89 mm (1 to 3.5 inches), and a 28 day minimum compressive strength of 25.17 MPa (3650 psi). Cylinders from both pours were taken and tested and the average compressive strength obtained after testing the full-scale models was 37.10 MPa (5380 psi).

Calculations indicated that the existing exterior columns on the I-80 overpass had an average of 890kN (200 kip) of dead load. Creating this axial load on the test specimens was accomplished using a single 159 mm (6.25 inch) diameter hydraulic ram with 68.95 MPa (10,000 psi) capacity and 152 mm (6 inches) of stroke. This ram acted vertically against a reaction beam on the top of the inverted column as shown in Figure 2. The reaction beam was attached to two saddle beams underneath the test specimen by means of eight pre-stressing cables. The hydraulic pressure was maintained at 44.96 Mpa (6,520 psi) throughout the testing sequence in order to sustain the 890 kN (200 kip) of axial dead load.

Instrumentation and Testing

In order to monitor the behavior of the beam-column joint under loading, four 120 Ohm strain gages were placed on reinforcing bars for each column test along the center line of the pier at the positions shown in Figure 2. Strain gages were also placed at various positions on the composite overlays after they were in place. Displacement transducers were placed at the top, middle, and bottom of each column as well as the end of the haunch and the center point on the cap beam between the supports. In addition to this instrumentation, a pressure transducer was calibrated and placed on the hydraulic pump running the three parallel hydraulic rams used to test the joint as shown in Figure 2. Data from all instrumentation was collected at 1 second intervals throughout the testing sequence and displayed on a laptop computer screen using data acquisition software.

The horizontal loading in the specimens was applied pseudo-dynamically, using the three parallel hydraulic rams, each with a 152 mm (6 inch) bore, 20.68 MPa (3,000 psi) capacity, and an 457 mm (18 inch) stroke. These rams acted at an angle of 45° to the column, as shown in Figure 2, and were attached to the column and pier via the structural steel embedments placed inside the re-bar cage prior to placing the concrete.

The test of the first beam-column joint failed due to local yielding in the reaction beam and saddle beams on the column dead load apparatus and the 890 kN (200 kip) dead load was not maintained during this test. Therefore, no pertinent data was acquired from this test.

In the subsequent five tests the testing sequence followed procedures similar to Ghobarah, Aziz, and Biddah (1996). A load-controlled phase, consisting of two cycles each at 89, 178, 267, and 356 kN (20, 40, 60, and 80 kips), was applied to cause initial cracking of the concrete. Three cycles were then applied at the theoretical yield point of the extreme steel in the beam-column joint. This load was measured to be approximately 445 kN (100 kips). The displacement corresponding to first yield in the steel was then recorded and used in the displacement controlled phase of the testing sequence. Three cycles each at 50% increments of the first yield displacement were applied until the specimen was only able to resist 75% of the ultimate load.

Analysis and Results

The six tests are listed below. The data from the second test, the successful test of a bare concrete specimen, was used as a benchmark for comparison to the rest of the tests. The test number and description are:

Test #1: Bare Concrete (As-Built)

Test #2: Bare Concrete (As-Built)

Test #3: Composite - 45° Composite Wraps (Figure 4)

Test #4: Composite - 30° Composite Wraps

Test #5: Composite - Column Composite Wrap Only

Test #6: Composite - 0° and 45° Composite Wraps

The deflections of the specimens at yield and ductilities of each beam-column joint, μi, were calculated for each loading direction using methods similar to those utilized by Paulay and Priestly (1992). The ratio of μi to $\mu 1$ shows, in most cases, an increase in ductility of the beam-column joints utilizing a composite wrap over the bare unaltered concrete specimen.

The results of this testing program are summarized are summarized in the following discussion and in Table 1.

Test #1: Bare Concrete (As Built)

This initial test failed due to the inability to maintain the 890 kN (200 kip) axial load in the column.

Test #2: Bare Concrete (As Built)

The bare concrete specimen provides a basis for comparison. The hysteresis diagram for the loading history of this test is given in Figure 3. This diagram shows that this specimen has some, though not a lot, ductile behavior. The maximum horizontal load, Pult, carried by the as-built specimen reached 525 kN (118 kips) in the push direction and 636 kN (143 kips) in the pull direction. The as-built specimen reached total ductility levels, $\mu 2$, of 3.10 in the push direction and 3.36 in the pull direction at deflections, $\Delta 2$, of 79 mm (3.11 inches) and in push and 59 mm (2.32 inches) in pull before failure.

The column and cap beam experienced minor fatigue cracks acting horizontally in the column and vertically in the cap beam at horizontal loads of 267 to 356 kN (60 to 80 kips). Yielding occurred in the longitudinal re-bar on the compression face of the beam during the first push cycle of 445 kN (100 kips). Shear cracks increased in the column/cap joint at levels of loading above 356 kN (80 kips). Spalling of concrete occurred at the base of the column as the vertical re-bar began to yield and push the concrete out at the column-cap interface.

Test #3: 45° Composite Wraps with Column Composite Wraps

The first of the composite wrapped specimens is shown in Figure 4. Single layers of 1.07 m (42 inch) wide, unidirectional, carbon-fiber composite pre-preg sheets were wrapped in an "ankle wrap" fashion around the column-cap joint at 45° angles to the horizontal. Additional wraps were wrapped at 90° around the beam and haunch of the bent cap next to the column, overlapping the 45° wraps. These were in addition to the wrapping of the column with composite material. The hysteresis diagram for the testing of this specimen is plotted in Figure 5. This specimen was one of the two most ductile specimens tested and the hysteresis loops of this diagram show more ductile behavior when compared to the hysteresis curves from the testing of the as-built specimen (Test #2). As indicated by this plot, improvements were shown in both ductility and strength in both the push and pull directions. Total ductilities, $\mu 3$, of 4.64 and 3.58 were achieved at deflections, $\Delta 3$, of 121 and 72 mm (4.76 and 2.82 inches) and ultimate loads of 578 and 685 kN (130 and 154 kips) in the push and pull directions, respectively.

Test #4: 30° Composite Wraps with Column Composite Wrap

The 30° wraps were placed on the bent-cap in a fashion similar to that of the 45° wraps with the exception of the angle of the wraps to the horizontal plane.

The ultimate loads were 605 and 685 kN (136 and 154 kips), the ductility, μ_4, was 3.99 and 4.48, and the deflections were 108 and 90 mm (4.27 and 3.55 inches) in the push and in the pull directions.

Test #5: Column Composite Wrap Only

The composite wrap for this specimen consisted of wraps on the columns only, in the same manner that the previous two specimens were wrapped on the column. Failure occurred in the push direction at a load of 538 kN (121 kips) with a total deflection, Δ_5, of 72 mm (2.85) inches corresponding to an overall ductility, μ_5, of 2.85. Failure in the pull direction occurred at a load of 649 kN (146 kips) at a deflection of 84 mm (3.29 inches) of deflection with an overall ductility of 3.85.

The failure mechanism followed the same pattern established by the as-built column with the exception of the deterioration at the base of the column. The composite wrap proved very successful in this region of the specimen. The only noticeable damage occurred in the two inch gap between the base of the composite wrap and the cap beam where cracks appeared in the column concrete. The failure of the joint area was very similar to the as-built design with shear cracks throughout the joint region.

Test #6: 0° and 45° Composite Wraps with Column Composite Wraps

This design is identical to the 45° wrap but with an additional 0° layer of composite applied directly to either side of the cap-beam, just under the 45°wrap, in the column-cap joint region.

Although some improvement was made over the as-built model with this design, the 0° and 45° wrap with ductility levels, μ_6, of 3.73 in the push direction and 3.50 in the pull, fell short in comparison to the specimen with 45° wraps and no 0° wraps which had a ductility levels, μ_3, of 4.64 and 3.58 in the same directions. The ultimate loads and deflections were 605 and 685 kN (136 and 154 kips) and 96 and 67 mm (3.77 and 2.62 inches) in the push and pull directions, respectively.

CONCLUSIONS

The small number of tests performed, limited by the size and expense of full-scale testing, makes it difficult to draw conclusions that are absolute and statistically reliable. However, several observations were made from these tests. First, wrapping of the columns forced the failure into the bent-cap. This is in contrast to the bare concrete specimen (Test #2) where the failure occurred in the joint of the column and bent-cap with each element experiencing severe damage. Second, the specimen with only the column wrap (Test #5) experienced a premature failure due to the de-bonding of the longitudinal column reinforcement within the bent-cap. Third, the 0° layer of composite on the bent-cap, which was bonded directly to the concrete (Test #6), appears to have inhibited the effectiveness of the 45° "ankle wrap" which was wrapped over the 0° composite layer. Fourth, it is unclear which of the ankle wraps, the 30° or 45°, was more effective. However, in general, wrapping the bent-caps and the columns did provide an increase in the ductility of the column/bent-cap joint by factors up to 1.5.

The stiffness and strength of the wrapped specimens do not increase over the bare one as significantly as the ductility does. This was expected to be the case. A layer of graphite composite, even with a very high tensile strength, will not contribute significantly to the strength or stiffness of an extremely massive and rigid system like a reinforced concrete beam-column joint.

ACKNOWLEDGEMENTS

The authors would like to thank the Priority Technologies Program (PTP) of the Federal Highways Administration (FHWA), Utah Department of Transportation (UDOT), and the Utah Transportation Center (UTC) for funding this research.

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