RETROFIT OF EXISTING CONCRETE BEAM-COLUMN JOINTS USING ADVANCED CARBON-FIBER COMPOSITES

Marvin W. Halling, Ph.D., P.E.
Assistant Professor

Kevin C. Womack, Ph.D., P.E.
Associate Professor

Robert M. Moyle, M.S.

Department of Civil and Environmental Engineering
Utah State University
Logan, Utah

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Disclaimer

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INTRODUCTION

In May 1996, the Rebuild America Coalition reported, “One out of three bridges in America is rated structurally deficient or functionally obsolete and needs major improvements ranging from deck replacements to complete reconstruction. More than a fourth of all bridges are more than 50 years of age, which is the average life span of a bridge. The cost to eliminate the backlog of existing deficiencies is approximately $78 billion.”

For the past several years, seismic design of structures has been continuously upgraded to meet higher seismic resistance standards. According to today’s standards, bridges built before 1971 are considered seismically deficient. This problem is compounded in Utah where freeze/thaw cycles are frequent and cause the outer protective layer of concrete on bridge structures to spall and expose the reinforcing bars. The problem is then complicated by the use of road salt, which is spread on all major roadways to eliminate snow and ice. The salt used on the roadways and bridges chemically reacts with the exposed steel in the reinforced concrete and accelerates corrosive damage to the steel. The poor condition of the bridges along the Wasatch Fault, which runs directly through the major population centers of the state — namely Salt Lake, Weber, Utah, Davis, and Box Elder counties — make Utah highly vulnerable to significant structural damage of bridges during an even moderate seismic event.

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

Research at the University of California at San Diego (UCSD) (Seible, Hegemier, Priestley, Innamorato, Weeks, and Policelli, 1994) has shown that the performance of non-ductile concrete columns retrofitted with various composite wraps is comparable to that of steel jacketing. 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.

ADVANCED CARBON-FIBER COMPOSITE RETROFIT STUDY

UDOT, in conjunction with researchers from Utah State University and the University of Utah, began in late 1995 to investigate the application of the advanced carbon-fiber composites to a bridge pier cap and column joint. The Interstate 80 overpass crossing Highland Drive in Salt Lake City was chosen as a model for the investigation. A plan and elevation view of the overpass at Interstate 80 (I-80) and Highland Drive are shown in Figure 1. This bridge was chosen for several reasons: 1) The bridge was in need of major repair due to spalling of concrete and severe decay of the exposed reinforcing bars in the piers, 2) It had already undergone a partial retrofit of an additional six inches of concrete and additional steel reinforcing on pier #4 (see Figure 1) of the east- and west-bound lanes (this will provide a comparison to the composite retrofit), 3) The bridge is scheduled for replacement in 10 to 15 years, at which time the bridge may be available for full-scale destructive testing.

To complete the analysis of placing an advanced carbon-fiber retrofit on this structure, it was proposed that the project be divided into three main areas. The University of Utah would perform computer modeling of the existing structure and design the composite overlays. XXsys Corporation of San Diego, California would apply the composite overlays to the full-scale test specimens and to the existing bridge. Utah State University would perform the design, fabrication, and physical testing of full-scale specimens to validate the ultimate capacities and ductilities of the advanced carbon-fiber retrofit schemes.

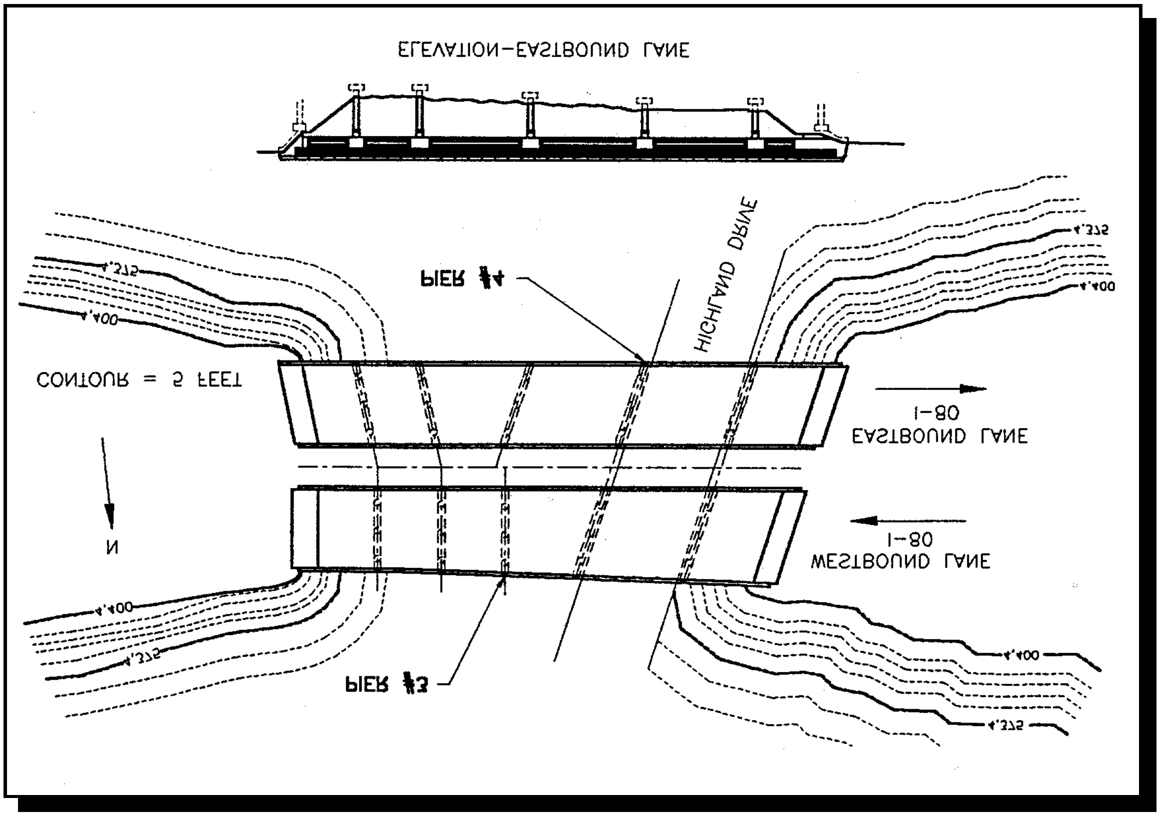

Figure 1 Plan and Elevation view of the Interstate 80 and Highland Drive overpass in Salt Lake City, Utah.

This report outlines the work done by the research team at Utah State University in designing, building, testing, and analyzing data from six full-scale test specimens based on the design of pier #3 on the westbound lane of I-80 crossing Highland Drive. A schematic of pier #3 is shown in Figure 2.

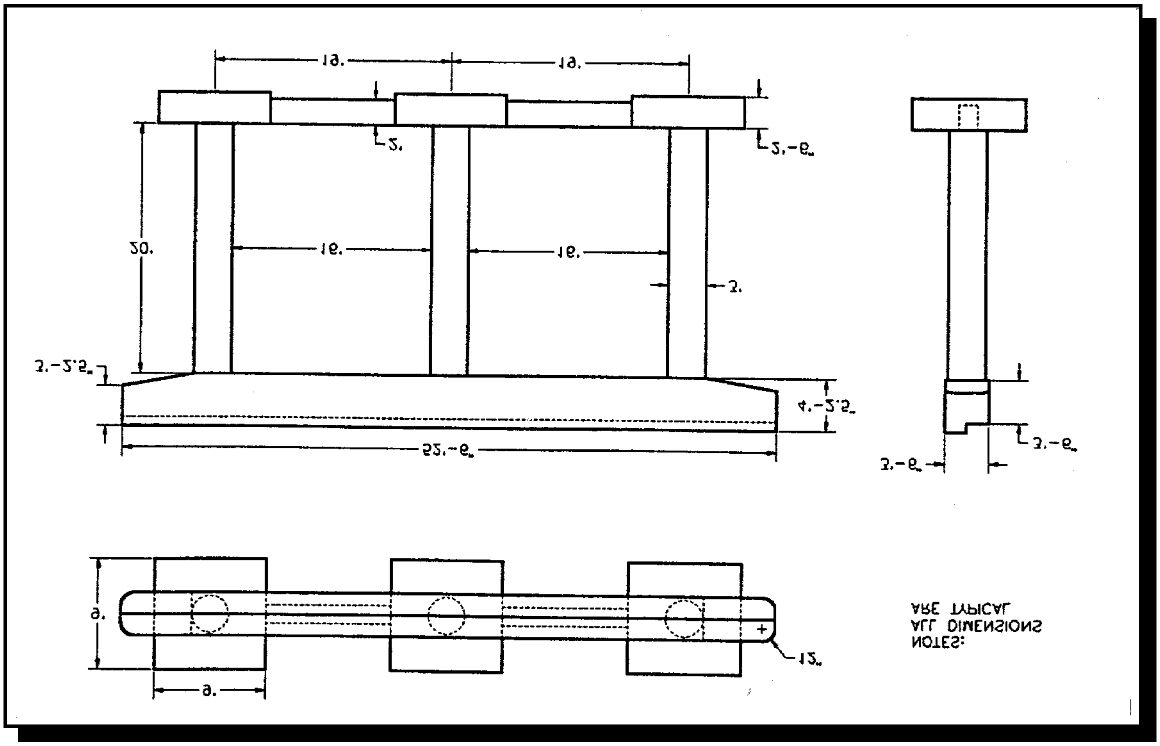

Figure 2 Schematic view of Westbound Pier #3. (Scale 1:200).

FULL-SCALE MODELING

The physical testing of six full-scale models was accomplished at Utah State University in 1996. Two of the 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 then applied to the remaining four models to test the effectiveness of various retrofit designs using the 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 four major components: design and construction, instrumentation and testing, analysis and results, and conclusions.

DESIGN AND CONSTRUCTION

Several design ideas were considered and a self-contained configuration was chosen as the design for the test specimen shown in Figure 3. Note that this configuration actually is an inverted replica of the existing bridge pier without the center column. The models 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. The supports were built 15 inches high to accommodate flexure of the beam under loading and to allow enough room for workers to place composite wraps under the specimens.

The vertical and longitudinal reinforcing bars in the column and beams of the test specimens were modified from Grade 40 rebar, which was used in the existing structure, to Grade 60 rebar, which is more readily available today. An equivalent amount of Grade 60 reinforcing bars for the model was equated to the actual amount of Grade 40 steel in the existing structure based on the following ratio:

$$A_s^{(60)}F_y^{(60)} = A_s^{(40)}F_y^{(40)}$$

Where $A_s^{(60)}$ and $A_s^{(40)}$ are the areas of the Grade 60 and Grade 40 reinforcement steel respectively, and

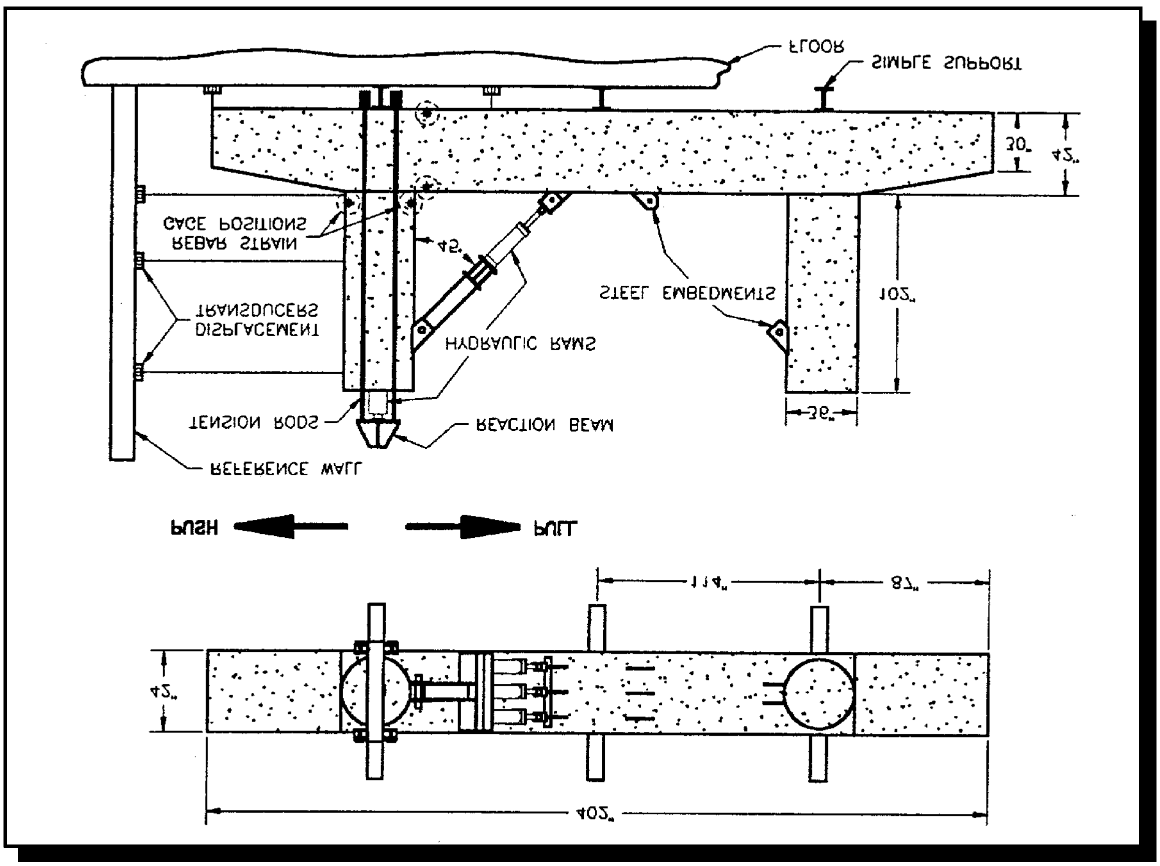

Figure 3 Full-scale testing configuration at Utah State University. (Scale 1:110).

$F_y^{(60)}$ and $F_y^{(40)}$ are the yielding stresses for Grade 60 and Grade 40 steel respectively. The development lengths in the reinforcing bars also were equated according to individual size and grade. The modified specimens called for 13 No.8 Grade 60 vertical bars in each column, six No. 7 Grade 60 longitudinal bars in the top of the inverted bent-cap, 11 No.6 Grade 60 longitudinal bars in the bottom of the inverted bent-cap, two No.4 Grade 60 longitudinal bars on each side of the bent-cap, and six No. 4 Grade 60 longitudinal bars on the top of the inverted haunch, all at equal spacing with two inches of concrete cover. The hoops and stirrups used were Grade 40 steel, which maintained the same size bars and spacing as the existing bridge pier.

To simulate stirrup deterioration 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, as shown in Figure 4.

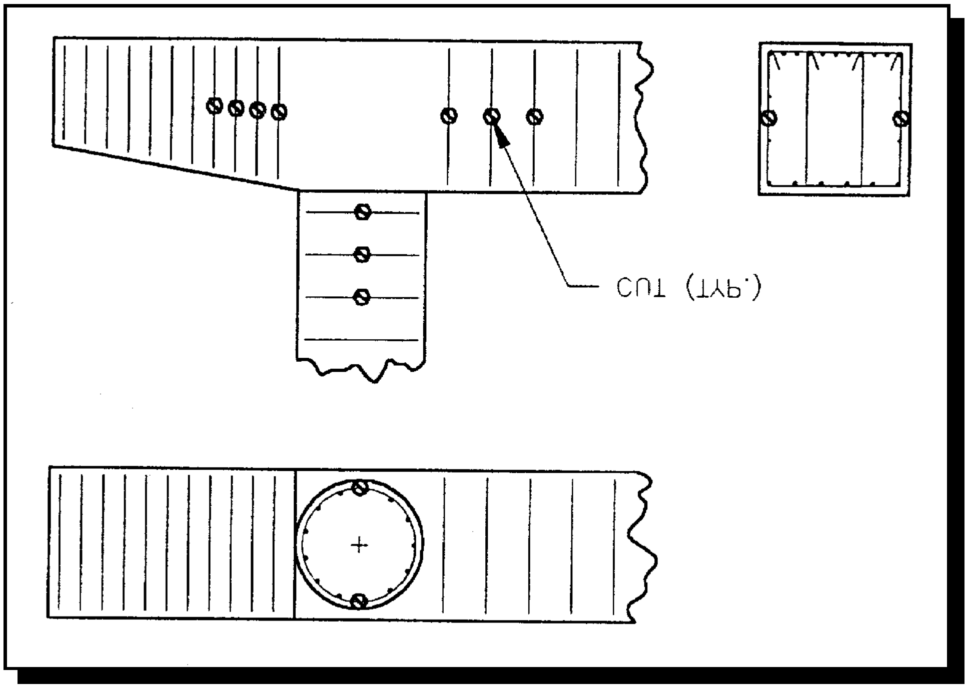

Figure 4 Exposed view of stirrups showing cut positions. (Scale 1:60).

To accommodate the loading apparatus used to test the model, structural steel embedments were designed, assembled, and placed in the mesh-work of the reinforcing cage in the center of the cap beam and at the top of each column as shown in Figure 3. The steel embedment in the center of the cap beam was welded to three 20-foot long No.11 Grade 60 reinforcing bars placed in the center of the cap beam. The bars anchored the embedment and reinforced the concrete against forces created during the testing. The column embedment was welded to five 5-foot long No. 11 Grade 60 reinforcing bars to prevent the push out of the embedment by the vertical force component of the rams. Minimal adjustments were made to the existing rebar cages to accommodate the embedments.

Concrete for the models was poured in two separate sequences. The beams and haunches were poured first on March 15, 1996, and the columns were poured April 18, 1996. This sequence created a cold joint at the column and beam interface similar to the one on the existing I-80 structure. Concrete used in the model was a UDOT Class-AA mix. This calls for a course aggregate size from one inch to no. 4 sieve, a minimum cement content of 6.5 sacks per cubic yard, a total slump of one to 3.5 inches, and a 28-

day minimum compressive strength of 3650 psi. Cylinders from both pours were taken and tested and the average compressive strength obtained after testing the full-scale models was 5380 psi.

Calculations indicated that the existing exterior columns on the I-80 overpass had an average of 200 kips of dead load. Creating this axial load on the test specimens was accomplished using a single 6.25 inch diameter hydraulic ram with 10,000 psi capacity and six inches of stroke. This ram acted vertically against a reaction beam on the top of the inverted column as shown in Figure 3. 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 6,520 psi throughout the testing sequence in order to sustain the 200 kips of axial dead load.

INSTRUMENTATION AND TESTING

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 3. Strain gages also were 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 3. Data from all instrumentation was collected at 1 second intervals throughout the testing sequence and displayed on a laptop computer screen using DaqView v. 4.2 data acquisition software.

Horizontal loading in the specimens was applied pseudo-dynamically, using three parallel hydraulic rams, each with a six-inch bore, 3,000 psi capacity, and an 18-inch stroke. The rams acted at an angle of 45° to the column, as shown in Figure 3, and were attached to the column and pier via the structural steel embedments placed inside the rebar cage prior to placing the concrete.

Testing commenced on Aug. 1, 1996, when the first of six columns was tested to failure. Due to local yielding in the reaction beam and saddle beams on the column dead load apparatus, the 200 kip dead load was not maintained during this test. Therefore, the data acquired is not included in this report as it does not reflect a direct comparison to the five remaining tests where the axial load of 200 kips was maintained and monitored in each column.

The testing sequence followed procedures similar to Ghobarah, Aziz, and Biddah (1996). A load-controlled phase, consisting of two cycles each at 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 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 0.5 increments of the first yield displacement were applied until the specimen was only able to resist 75 percent of the ultimate load.

ANALYSIS AND RESULTS

The first successful test was performed on a bare column and bent-cap joint. This data was used as a benchmark for comparison to the rest of the tests. Therefore, the tests are numbered as follows:

Test #1: Bare Concrete (As-Built)

Test #2: Composite - Column Composite Wrap only. (Figure 7)

Test #3: Composite - 30° Composite Wraps (Figure 10)

Test #4: Composite - 45° Composite Wraps (Figure 13)

Test #5: Composite - 0° and 45° Composite Wraps (Figure 16)

Table 1 summarizes the test results from each beam-column specimen. P_{ult} represents the ultimate load in kips that each specimen achieved for each respective loading direction. Δ_i is the total deflection at failure for each specimen, *i,* in each loading direction. The ductility of each beam-column joint, μ_i, was calculated for each loading direction using similar methods to those used by Paulay and Priestley (1992). This method defines the division between the elastic and plastic ranges. μ_i / μ_1 is a ratio of the ductility of each test to the ductility of the as-built specimen.

Table 1. Summary of Test Results

	Push				Pull			
	P_{ult} (kips)	**Δ_i (in.)**	**μ_i**	**μ_i / μ_1**	**P_{ult} (kips)**	**Δ_i (in.)**	**μ_i**	**μ_i / μ_1**
Test #1	118.87	3.14	3.77	1.00	146.58	2.31	3.28	1.00
Test #2	120.84	2.89	3.47	0.92	147.81	3.47	4.91	1.50
Test #3	138.63	4.41	5.30	1.40	154.76	3.64	5.15	1.57
Test #4	130.26	4.67	5.61	1.49	154.93	2.92	4.14	1.26
Test #5	136.82	3.68	4.42	1.17	154.96	2.70	3.83	1.17

Test #1: Bare Concrete (As-Built)

To establish a basis for comparison, a bare concrete specimen was tested first. The loading history for this test is given in Figure 5. As seen from Figure 5, the maximum horizontal load, P_{ult}, carried by the as-built specimen reached 118.87 kips in the push direction and 146.58 kips in the pull direction. Figure 6 shows the load-deflection envelope for the as-built test, which reached total ductility levels, μ_1, of 3.77 in the push direction and 3.28 in the pull direction 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 60 to 80 kips. Yielding occurred in the longitudinal rebar on the compression face of the beam during the first push cycle of 100 kips. Shear cracks increased in the column/cap joint at levels of loading above 80 kips. Spalling of concrete occurred at the base of the column as the vertical rebar began to yield and push the concrete out at the column-cap interface.

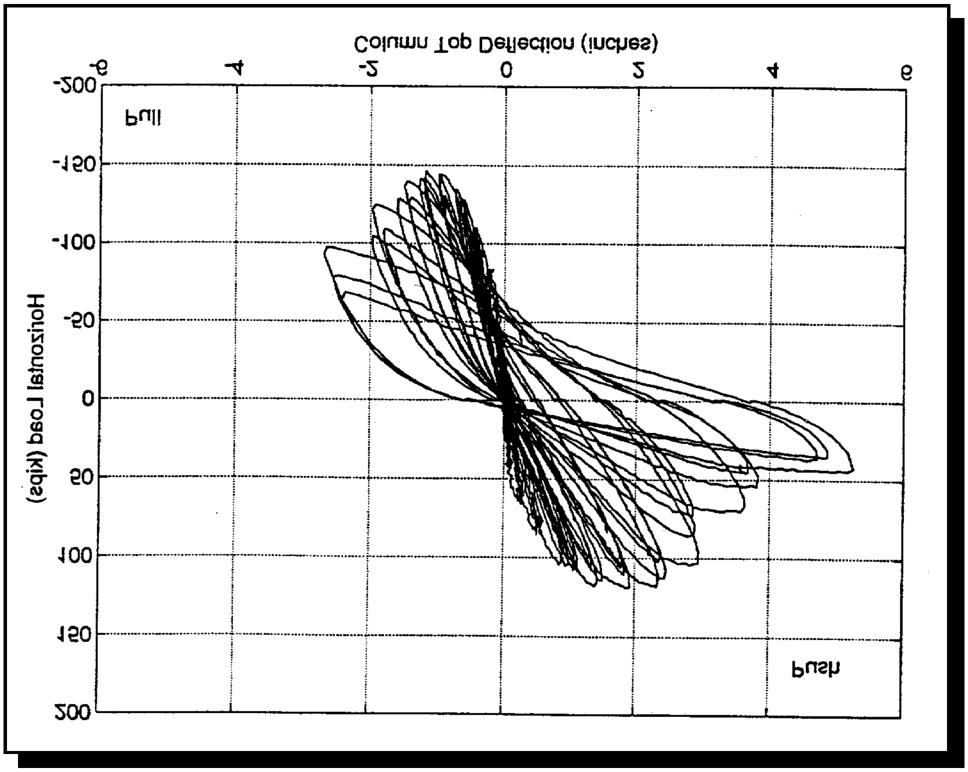

Figure 5 Test #1: Bare concrete load-deflection curve.

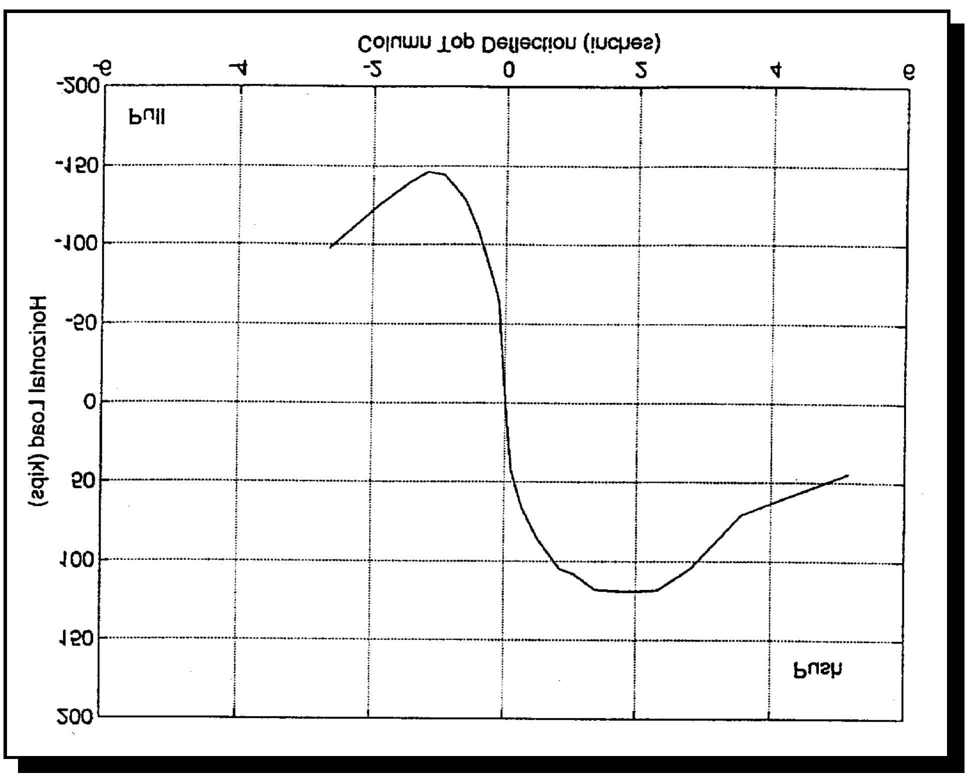

Figure 6 Test #1: Bare concrete load-deflection envelope.

Test #2: Column Composite Wrap Only

Figure 7 shows a schematic of the column wrap only test specimen. As shown, this retrofit design consisted of eight layers of pre-preg tow wraps placed on 18 inches of the column nearest the column-cap joint, and an additional four layers placed from 18 to 36 inches above the column-cap joint. The load-displacement curve is given in Figure 8. Failure occurred in the push direction at a total deflection, Δ_2, of 2.89 inches corresponding to an overall ductility, μ_2, of 3.47. Failure in the pull direction occurred at 2.31 inches of deflectio n with an overall ductility of 4.91.

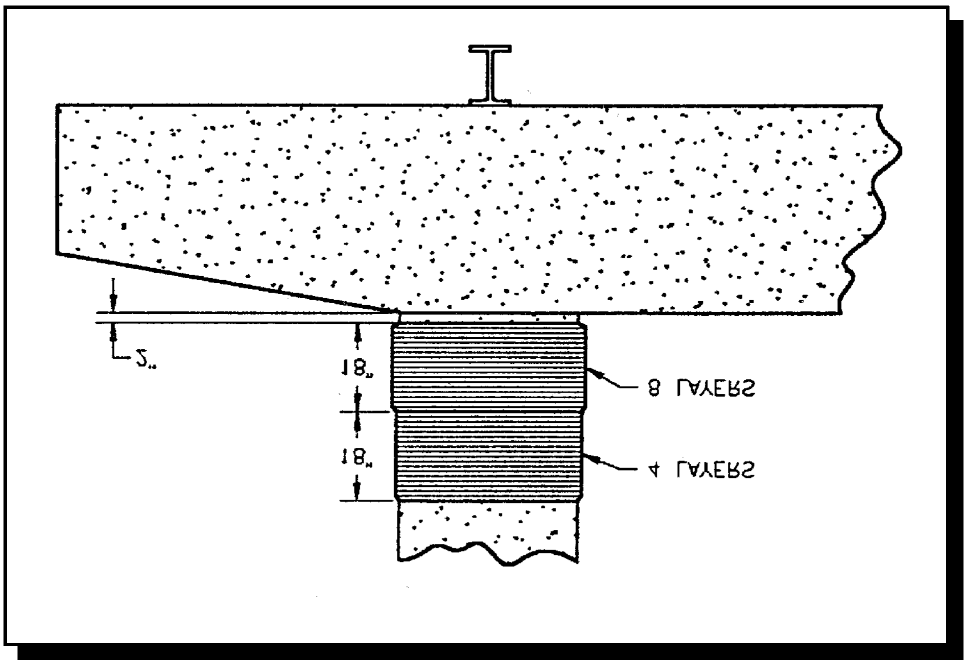

Figure 7 Test #2: Design for column composite wrap only. (Scale 1:50).

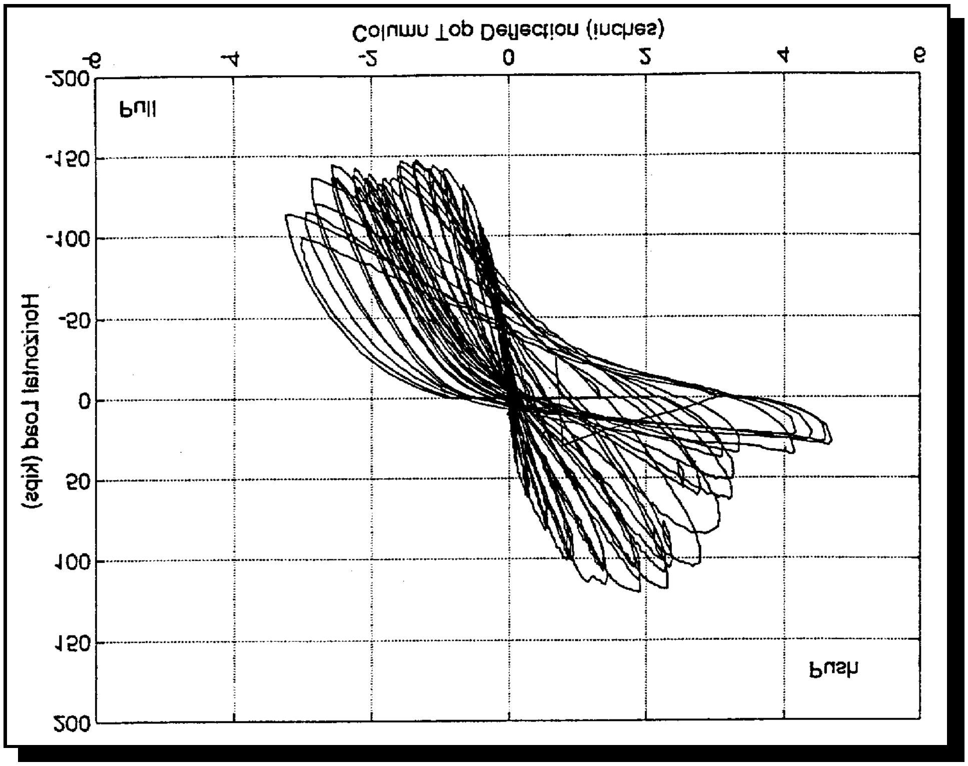

Figure 8 Test #2: Column wrap only load-deflection curve.

To make a direct comparison to the as-built design, the load-displacement envelope for the column wrap only test was superimposed on the load-displacement envelope for the as-built design. This comparison is given in Figure 9, which shows that an early failure developed for the wrapped column specimen in the push direction. This may be due in part to the de-bonding of the vertical column reinforcing bars in the joint area.

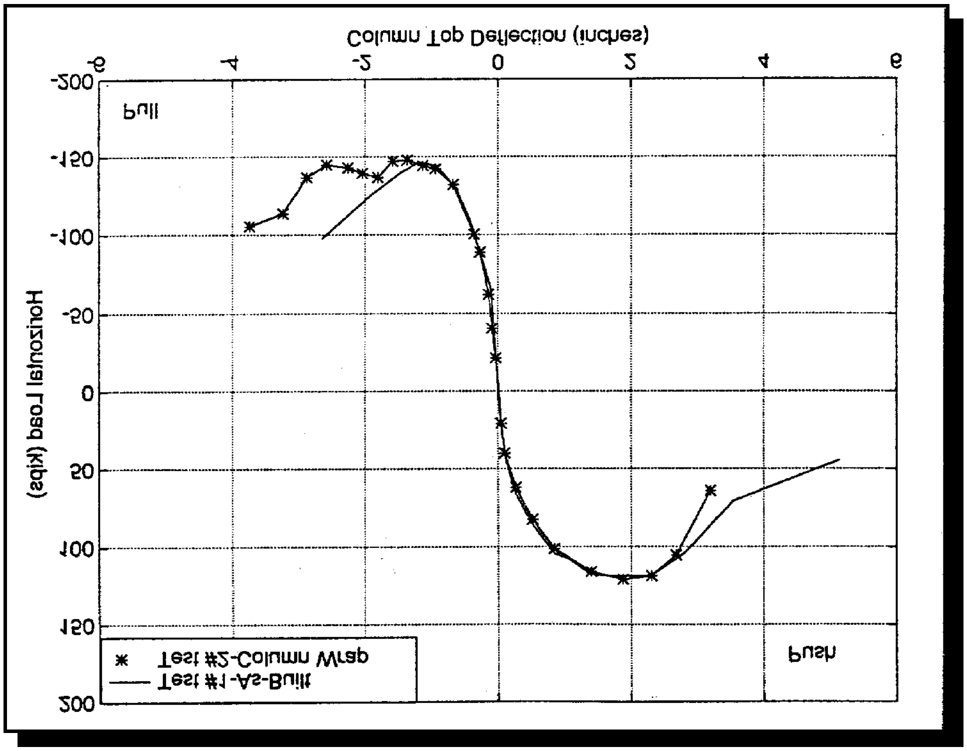

Figure 9 Comparison of Test #1 and Test #2 load-deflection envelopes.

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 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 similar to the as-built design with shear cracks throughout the joint region.

Test #3: 30° Composite Wraps with Column Composite Wrap

The first of three column and bent-cap joint wraps is shown in Figure 10. Single layers of 42-inch wide, unidirectional, advanced carbon-fiber composite pre-preg sheets were wrapped in an "ankle-wrap" fashion around the column-cap joint at 30° angles from the horizontal. Additional wraps were wrapped at 90° around the beam and haunch of the bent-cap next to the column, overlapping the 30° wraps. The column was wrapped identically to the column in the column wrap only design.

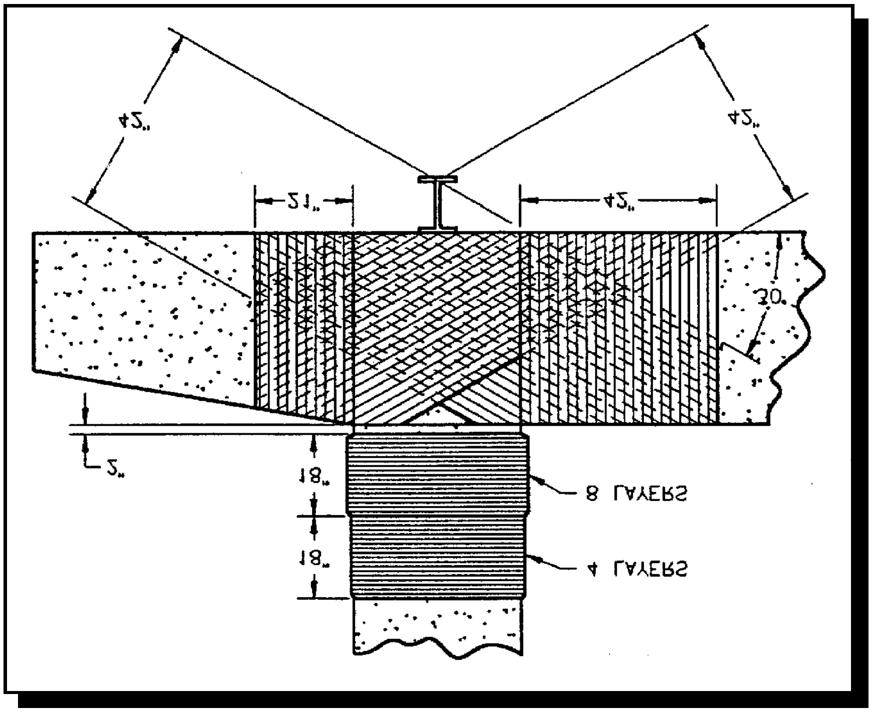

Figure 10 Test #3: Design for 30º composite wraps (Scale 1:50).

Figure 11 plots the load-deflection curve for this specimen. The ductility, μ_3, increased significantly to 5.30 in the push direction and 5.15 in the pull direction with a small increase in strength in both directions. Figure 12 compares the load deflection envelopes of the specimen with the 30° wraps to the as-built model.

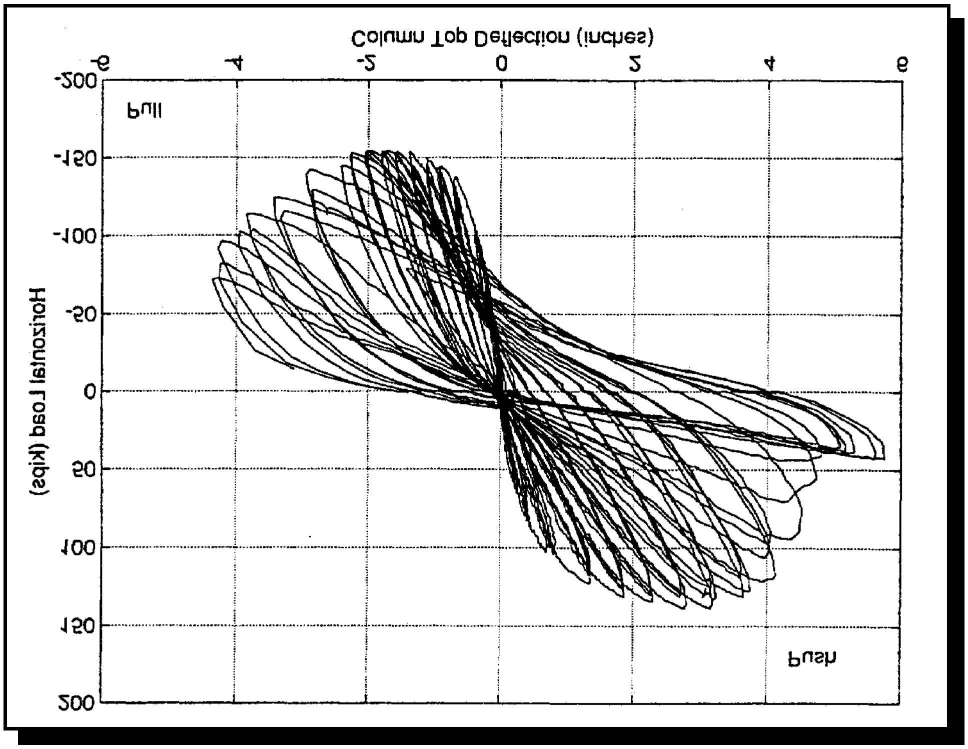

Figure 11 Test #3: 30° wrap load-deflection curve.

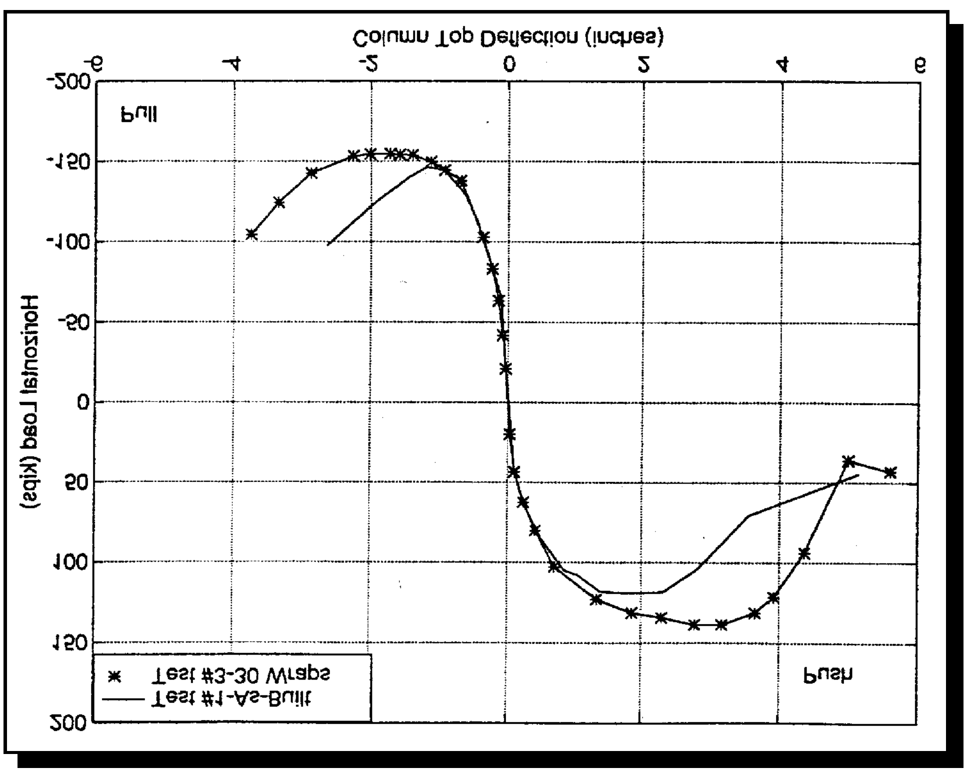

Figure 12 Comparison of Test #1 and #3 load-deflection envelopes.

Test #4: 45° Composite Wraps with Column Composite Wraps

As shown in Figure 13, the 45° composite wraps were placed on the bent-cap in a fashion similar to that of the 30° wraps with the exception of the angle. The column wrap was identical to the two

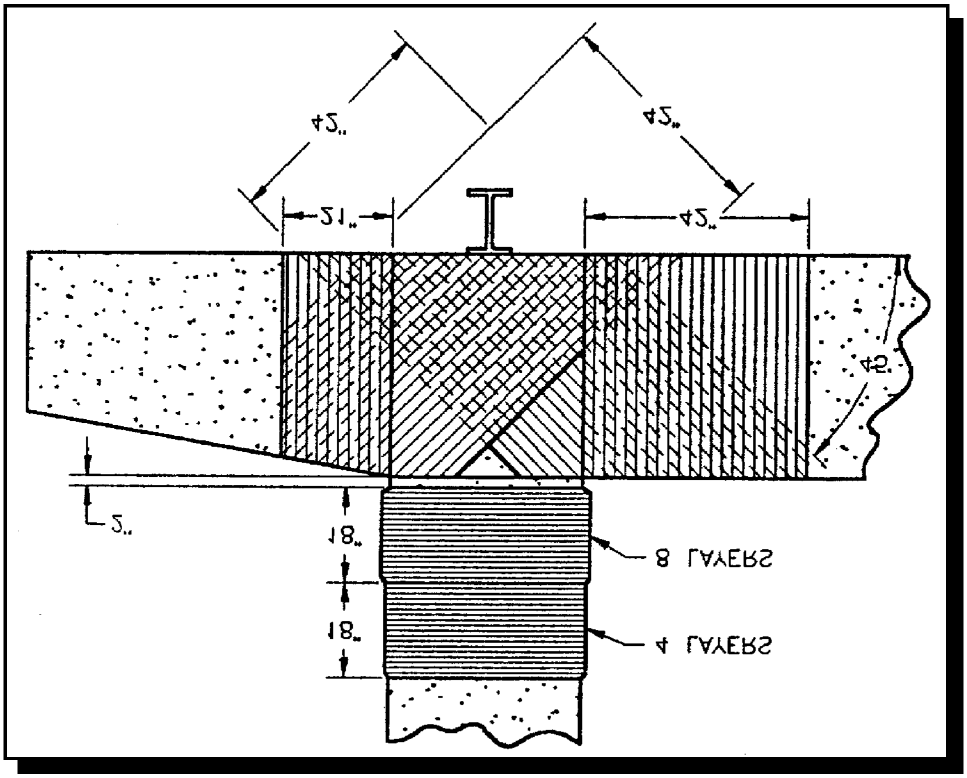

Figure 13 Test #4: Design for 45º composite wraps. (Scale 1:50).

previous tests.

The load-displacement curve for this test is plotted in Figure 14. As indicated by this plot, improvements were shown in ductility and strength in both the push and pull directions. Total ductilities, μ_4, of 5.61 and 4.14 were achieved at deflections, Δ_4, of 4.67 and 2.92 inches in the push and pull directions respectively. As shown in Figure 15 on the previous page, the 45° wrap maintained the greatest overall load-deflection envelope when compared to the previous three specimens.

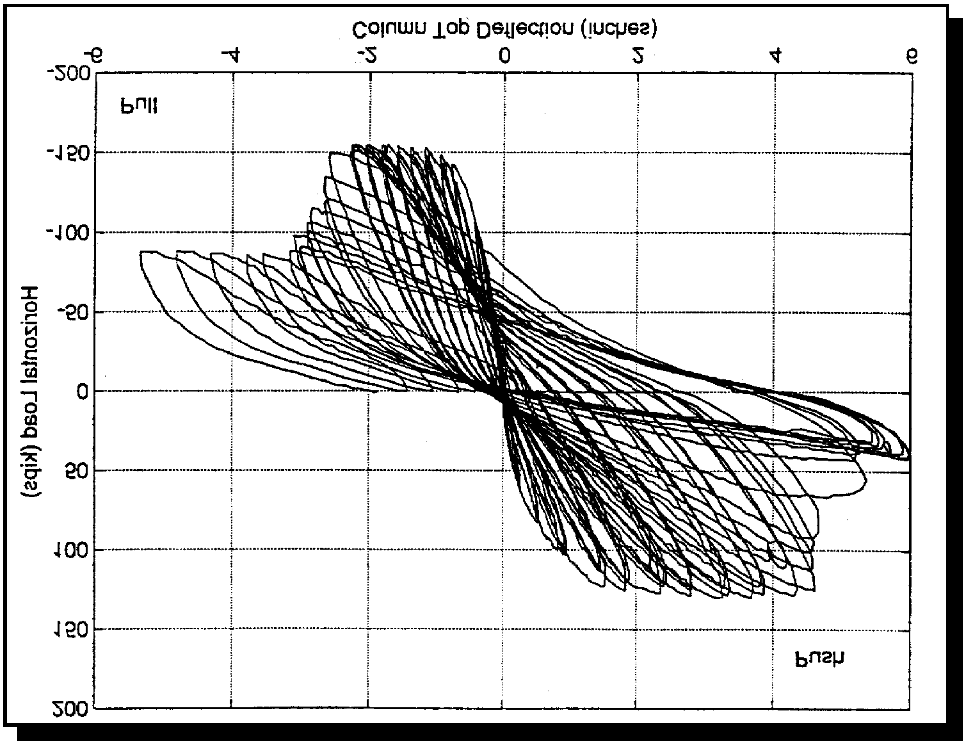

Figure 14 Test #4: 45º wrap load-deflection curve.

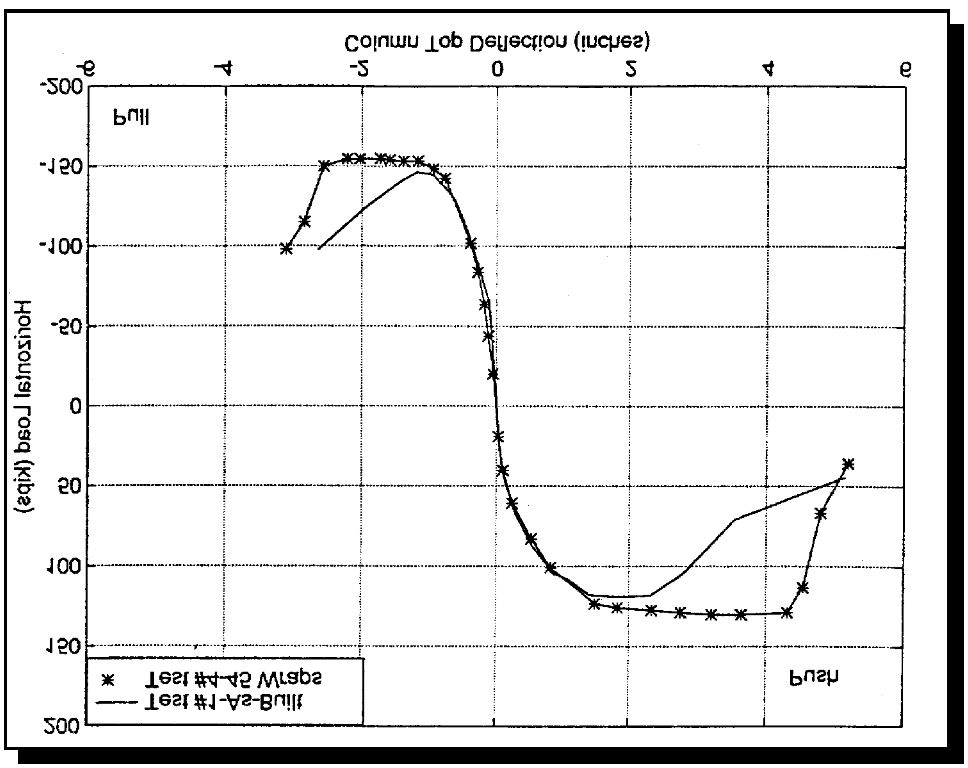

Figure 15 Comparison of Test #1 and Test #4 load-deflection envelopes.

Test #5: 0° and 45° Composite Wraps with Column Composite Wraps

Figure 16 is a schematic of the final 0° and 45° wrap design. This design is identical to the 45° wrap but with an additional 0° layer of composite applied to either side of the cap-beam in the column-cap joint region.

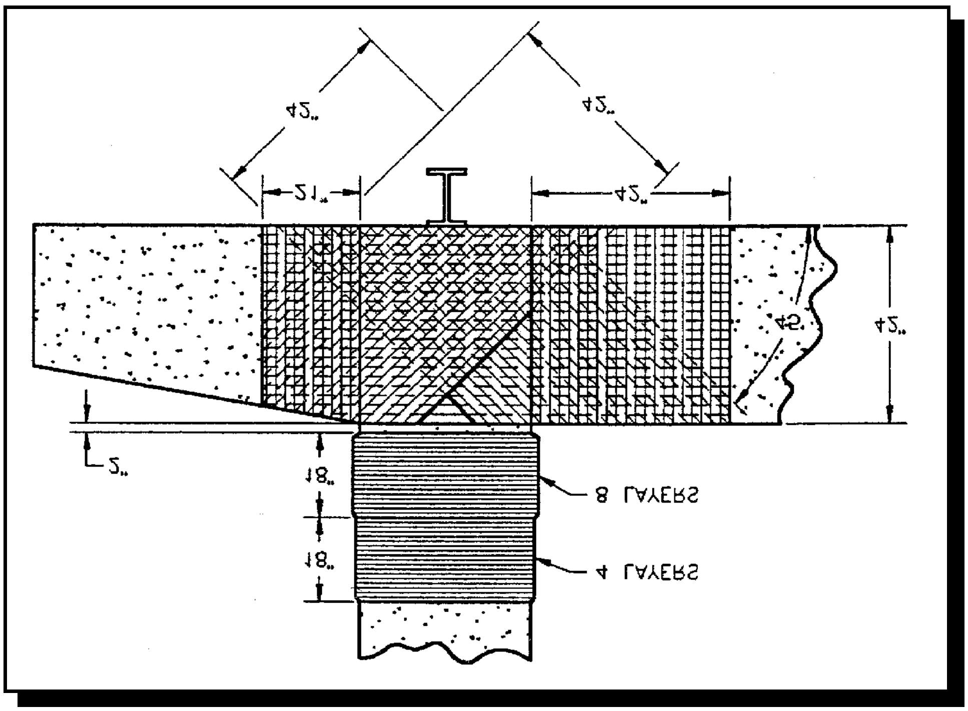

Figure 16 Test #5: Design for 0º and 45º composite wraps. (Scale 1:50).

The load-deflection curve is given in Figure 17. Figure 18 shows a comparison of the load-deflection envelopes between the 0° and 45° wrapped specimen and the as-built design. Although some improvement was made over the as-built model with this design, the 0° and 45° wrap with a ductility, μ_5, level of 4.42 in the push direction, fell short in comparison to the specimen with 45° wraps and no 0° wraps, which had a ductility level, μ_4, of 5.61 in the same direction.

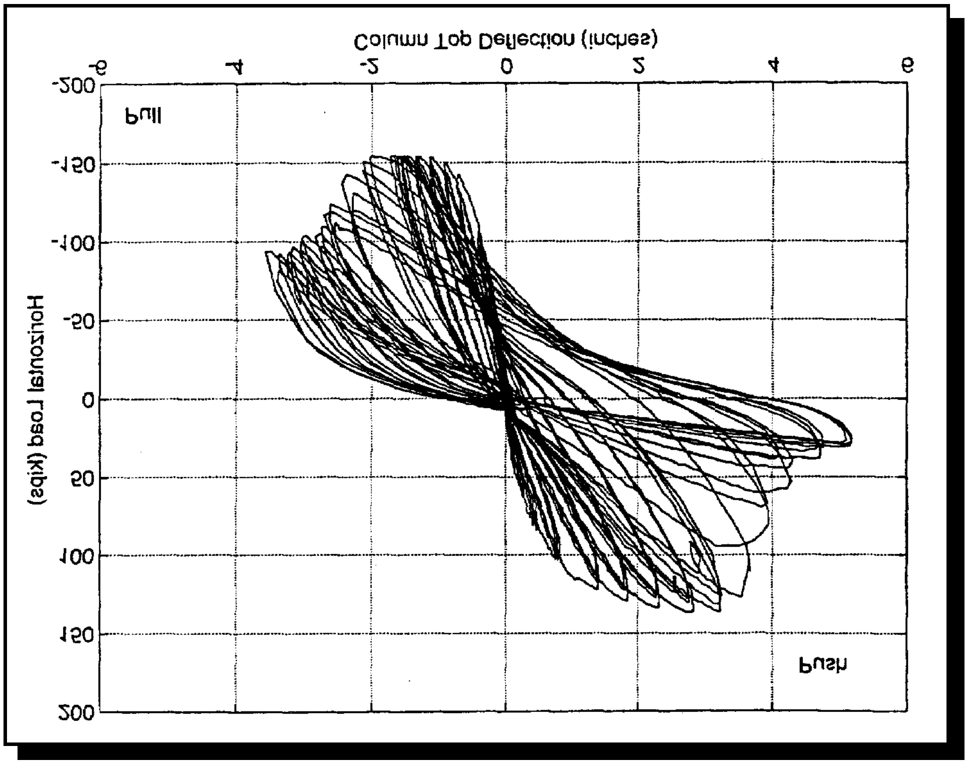

Figure 17 Test #5: 45° and 0° wrap load-deflection curve.

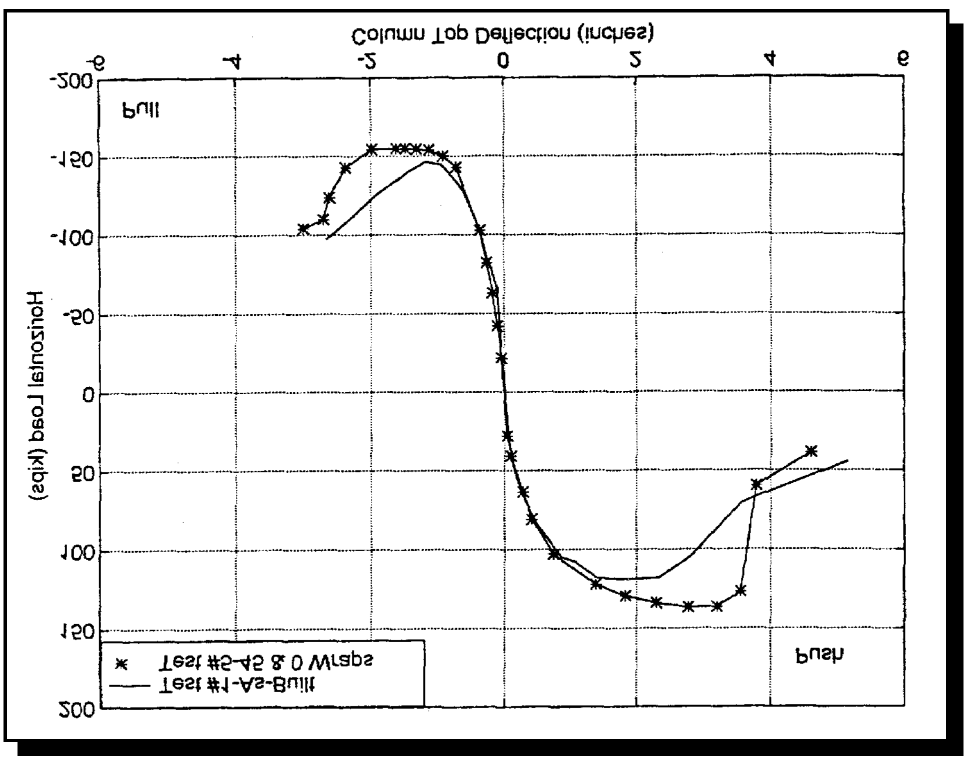

Figure 18 Comparison of Test #1 and Test #5 load-deflection envelopes.

CONCLUSIONS

The small number of tests performed, limited by the size and expense of full-scale testing, make it difficult to draw conclusions that are absolute and statistically reliable. However, several observations were made from the tests. First, wrapping of the columns forced the failure into the bent-cap. This is in contrast to the bare concrete specimen (Test #1) 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 #2) 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 #5), 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 a slight increase in strength and increased the ductility of the column/bent-cap joint by factors up to 1.5.

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