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CHAPTER 6

VIRGIN BEAM SUBASSEMBLY EXPERIMENTAL RESULTS

As described in Chapters 3, 4, and 5, a total of four precast concrete test beam specimens, referred to as Beams 1 to 4, were produced for eight floor level subassembly tests (Test 1 through 4B). As shown in Table 3.2, Beams 1 and 2 were each tested only once, whereas Beams 3 and 4 were re-used in a series of tests to investigate different parameters.

This chapter presents the results from Tests 1 – 4 in which Beams 1 to 4, respectively, are tested for the first time. These tests are referred to as the “virgin” beam tests in this dissertation. The chapter is divided into the following sections: (1) overall test specimen response; (2) Test 1; (3) Test 2; (4) Test 3; and (5) Test 4.

The results from the post-virgin beam tests are presented in Chapter 7, with a comparison, summary, and overview of all subassembly experimental results given in Chapter 8.

6.1 Overall Test Specimen Response

Photographs of the displaced subassembly at a beam chord rotation, \( \theta_b = 3.33\% \) from Tests 2 and 4 are shown in Figure 6.1. This rotation corresponds, approximately, to the FEMA 356 survival limit for monolithic cast-in-place concrete beams with diagonal
reinforcement, and is used as a comparative measure for the behavior of the precast concrete coupling beams in this dissertation (FEMA 2000).

Figure 6.1: Displaced position of subassembly at $\theta_b = 3.33\%$ – (a) Test 2; (b) Test 4.

The primary differences between Tests 2 and 4 are increased beam depth and decreased angle strength (see Table 3.2). Figure 6.2 shows the coupling beam shear force versus chord rotation ($V_b-\theta_b$) behaviors from the two tests up to a chord rotation of 3.33%. The results demonstrate the important hysteretic response characteristics of unbonded post-tensioned precast concrete coupling beams through self-centered hysteresis loops with moderate energy dissipation. The lateral strength of the structure and the relative amounts of self-centering and energy dissipation can be controlled by varying the beam geometry, post-tensioning tendon area, and angle strength.

According to ACI ITG 5.1 *Acceptance Criteria for Special Unbonded Post-Tensioned Structural Walls Based on Validation Testing* (ACI 2008), the ultimate “sustained” beam chord rotation is defined as the largest rotation that a beam is able to reach with not more than a 20% drop in shear resistance during three fully reversed
cycles. As shown in Figures 6.1 and 6.2, both test specimens are able to sustain 3.33% rotation with only a small amount of cracking and cover concrete spalling observed at the beam ends. The nonlinear lateral load behavior of the structure is governed by the opening of gaps at the beam-to-wall interfaces while the beam behaves similar to a rigid member. As the displacements of the structure are increased, the gap opening at the beam ends result in the yielding of the top and seat angles in tension. Upon unloading, the post-tensioning force pulls the structure back to its undisplaced shape, closing the gaps and yielding the angles back in compression resulting in a self-centered behavior (minimizing residual displacements). The yielding of the angles in tension and compression gives the system the energy dissipation that can be seen in the hysteresis loops in Figure 6.2. Note that other types of energy dissipation devices can also be used at the beam-to-wall interfaces, such as mild steel reinforcement, friction dampers, etc.; however, these are not investigated in this dissertation.

The following sections provide a detailed presentation of the results from Tests 1 – 4. The results for each test include: (1) test photographs; (2) beam shear force versus...
chord rotation behavior; (3) beam end moment versus chord rotation behavior; (4) beam post-tensioning forces; (5) angle-to-wall connection post-tensioning forces; (6) vertical forces on wall test region; (7) beam vertical displacements; (8) beam chord rotation; (9) local beam rotations; (10) load block displacements and rotations; (11) reaction block displacements and rotations; (12) contact depth and gap opening at beam-to-wall interfaces; (13) wall test region local concrete deformations; (14) beam looping reinforcement longitudinal leg strains; (15) beam transverse reinforcement strains; (16) beam confined concrete strains; (17) beam end confinement hoop strains; (18) wall test region confined concrete strains; (19) wall test region confinement hoop strains; and (20) crack patterns. The determination of these response parameters using the measured data is described in Chapter 5.

6.2 Test 1

The beam used in Test 1 (i.e., Beam 1) has the following properties: (1) beam depth, $h_b = 14$ in. (356 mm); (2) beam width, $b_b = 7.5$ in. (191 mm); (3) mild steel reinforcement of two No. 6 bars looping around the beam perimeter along its length; (4) No. 3 full-depth rectangular hoops [6.125 in. by 12.675 in. (156 mm by 322 mm)] placed at a nominal 7.0 in. (178 mm) spacing to provide transverse reinforcement in the beam midspan region; (5) No. 3 partial-depth rectangular hoops [6.125 in. by 4.375 in. (156 mm by 111 mm)] placed at a 1.5 in. (38 mm) spacing to provide concrete confinement at the beam ends; (6) a beam post-tensioning tendon comprised of two 0.6 in. (15 mm) nominal diameter high-strength strands with a total area of $A_{bp} = 0.434$ in.$^2$ (280 mm$^2$); (7) average initial beam post-tensioning strand stress of $f_{bpi} = 0.50 f_{bpu}$, where $f_{bpu} = 270$
ksi (1862 MPa) is the design maximum strength of the post-tensioning steel; (8) total initial beam post-tensioning force of $P_{bi} = 59.1$ kips (262 kN); (9) initial beam concrete nominal axial stress (based on actual cross-sectional area with beam post-tensioning duct removed) of $f_{bci} = 0.58$ ksi (4.0 MPa); and (10) two top and two seat angles (L8x8x1/2) with length, $l_a = b_b = 7.5$ in. (191 mm).

During construction, the south end of the beam and the wall test region of the reaction block had problems with the consolidation of the concrete. Due to the care taken to avoid damage to the embedded strain gauges during casting, the hoop confined areas of the beam and the wall test region were mostly void of concrete. A second reaction block was cast together with the casting of the remaining three beams (Beams 2 – 4) used in the experimental program. No consolidation problems occurred during this second batch of casting.

Prior to the casting of the second batch, the void regions of Beam 1 and of the first reaction block were patched with a high strength fiber reinforced grout for use in Test 1. This was done to determine any improvements needed in the test setup, instrumentation, and/or specimen design and construction for the subsequent tests. The patched regions of the reaction block and of the beam are shown in Figure 6.3. As a precautionary measure due to the patched regions, a relatively small amount of beam concrete initial stress was used in Test 1 (by using a relatively small amount of beam post-tensioning steel area).
6.2.1 Test Photographs

Photographs of the original and displaced subassembly configurations from Test 1 are shown in Figures 6.4 and 6.5. Figures 6.4(a) through 6.4(f) show overall subassembly photographs as follows: (a) pre-test undisplaced position; (b) displaced to $\theta_b = 3.0\%$; (c) displaced to $\theta_b = -3.0\%$; (d) displaced to $\theta_b = 8.0\%$; (e) displaced to $\theta_b = -8.0\%$; and (f) final post-test undisplaced position. Similarly, Figures 6.5(a) through 6.5(f) show close-up photographs of the south end of the beam at the beam-to-reaction-block interface. The accumulation of damage at the south end of the beam is shown in more detail in Figure 6.6.

The wall test region of the reaction block, including the patched region, did not receive any damage (no cracking and/or spalling of the concrete) during the test. Up
through a beam chord rotation, $\theta_b$ of 3.0%, the concrete cracking and spalling in the beam were also relatively small. As the beam chord rotation increased beyond 3.0%, the patched end of the beam suffered a significant amount of damage, which played an important role in the behavior of the subassembly as described later. The concrete at the unpatched (north) end of the beam performed very well throughout the test, with only a negligible amount of cover crushing at the corners.

The angle-to-wall connections performed well with no yielding in the connection strands and no damage to the wall concrete. The angle-to-beam connections also behaved well; however, a small amount of slip in the angle-to-beam connections between the angles and the beam was observed during the test at large rotation cycles, most likely due to the loss of force in the angle-to-beam connection bolts caused by the deterioration of the patched concrete at the south beam end. Slip between the coupling beam and the walls did not occur demonstrating that the friction resistance due to the post-tensioning force provided adequate vertical support to the beam together with the resistance from the top and seat angles. There were two unexpected premature wire fractures in the beam post-tensioning strands, occurring at beam chord rotations of 4.0% and 6.4%, as described later in more detail.

Under the displacements of the coupling beam up to about 3.0% rotation, gap opening occurred between the fiber-reinforced grout and the faces of the load and reaction blocks, which is the desirable mode of behavior for the structure as described in Chapter 3. Upon further displacements of the structure, the deformations at the patched end occurred primarily in the deteriorating beam concrete instead of gap opening [see Figure 6.7(a)]. The gap opening at the unpatched end continued to form until the end of
the test; however, it was observed that the heels of the top and seat angles came into contact with the grout column, causing the grout to buckle and separate from the beam end [Figure 6.7(b)]. Thus, it is recommended that the grout column is either tapered at the top and bottom or is terminated a little short of the full beam depth to prevent contact with the angle heels during large rotations of the beam. No significant crushing of the fiber-reinforced grout at the beam-to-wall interfaces was observed throughout the entire test.
Figure 6.4: Test 1 overall photographs –
(a) pre-test undisplaced position; (b) $\theta_b = 3.0\%$; (c) $\theta_b = -3.0\%$; (d) $\theta_b = 8.0\%$;
(e) $\theta_b = -8.0\%$; (f) final post-test undisplaced position.
Figure 6.5: Test 1 beam south end photographs –
(a) pre-test undisplaced position; (b) $\theta_b = 3.0\%$; (c) $\theta_b = -3.0\%$; (d) $\theta_b = 8.0\%$;
(e) $\theta_b = -8.0\%$; (f) final post-test undisplaced position.
Figure 6.6: Test 1 beam south end damage propagation – positive and negative rotations.

Figure 6.7: Test 1 gap opening and grout behavior at 8.0% rotation – (a) south end; (b) north end.
6.2.2 Beam Shear Force versus Chord Rotation Behavior

Figure 6.8(a) shows the hysteretic coupling beam shear force, $V_b$ versus chord rotation, $\theta_b$ behavior from Test 1, where $V_b$ and $\theta_b$ are calculated from Equations 5.5 and 5.35, respectively. The $V_b$-$\theta_b$ plot is terminated at a beam chord rotation of 5.0% when displacement transducer DT9 (see Chapter 5) was lost due to the spalling of the ferrule insert at the south end of the beam.

To show the complete hysteretic behavior of the subassembly from the entire test, Figure 6.8(b) plots the $V_b$-$\theta_{b,lb}$ behavior, where $\theta_{b,lb}$ is the beam chord rotation determined from the vertical ($y$-direction) displacement of the load block centroid using Equation 5.36 (see Section 6.2.8 for comparisons between $\theta_b$ and $\theta_{b,lb}$). It can be seen from Figure 6.8(b) that the subassembly was able to sustain three cycles at 6.4% rotation with approximately 11.5% loss in shear resistance. The ultimate failure of the structure occurred during the second cycle to 8.0% rotation as a result of the low cycle fatigue fracture of the horizontal leg of the seat angle at the unpatched (i.e., north) end of the beam (see Figure 6.9).

As shown in Figure 6.8, the specimen was able to dissipate a considerable amount of energy. Most of this energy dissipation occurred due to the yielding of the top and seat angles at the unpatched end. Angle yielding was limited at the patched end since the gap opening displacements remained small due to the patch deterioration at this end. Low cycle fatigue fracture was observed in the angles at the unpatched end of the beam. Despite a significant amount of loss in the post-tensioning force (see Section 6.2.4), the beam had a sufficient amount of restoring force to yield the tension angles back in compression and close the gaps at the beam ends upon unloading, resulting in a large self-centering capability.
Figure 6.8: Test 1 coupling beam shear force versus chord rotation behavior –
(a) using beam displacements; (b) using load block displacements.
6.2.3 Beam End Moment Force versus Chord Rotation Behavior

Figure 6.10 shows the hysteretic coupling beam end moment, $M_b$, versus chord rotation, $\theta_{b,lb}$ behavior from Test 1, where $M_b$ is calculated from Equation 5.6 as described in Chapter 5 and $\theta_{b,lb}$ is determined using the load block centroid displacements as described in the previous section. Since the beam end moment is calculated from the beam shear force, the results shown in Figure 6.10 are directly related to the results in Figure 6.8; and thus, no further discussion is provided herein.
6.2.4 Beam Post-Tensioning Forces

The coupling beam post-tensioning forces from the test are measured using load cells LC15 and LC16 mounted at the dead ends of the two strands (see Chapter 5). The forces from the two load cells, $F_{LC15}$ and $F_{LC16}$, are shown in Figure 6.11, and are plotted against the load block beam chord rotation, $\theta_{b,lb}$ in Figure 6.12. Figure 6.13 shows the total beam post-tensioning tendon force, $P_{bp}$ (sum of the forces in the two strands) normalized with the total design ultimate strength of the tendon, $\Sigma a_{bp} f_{spu}$. The total initial beam post-tensioning tendon force is equal to 59.1 kips (263 kN), resulting in an initial beam concrete nominal axial stress of $f_{bci} = 0.58$ ksi (4.0 MPa).
As the structure is displaced and gap opening occurs at the beam ends, the post-tensioning strands elongate and the post-tensioning forces increase. Since the strands are unbonded over the entire length of the subassembly, the nonlinear straining of the post-tensioning steel is significantly delayed. However, as shown in Figures 6.11 and 6.12, two sudden and unexpected drops are observed in the tendon force at 4.0% and 6.4% rotation; each of which occurred due to the premature fracture of a single wire (out of a total of seven wires) in a post-tensioning strand inside an anchor (similar to the wire fracture in Figure 4.7). The corresponding force drop in strands 1 and 2 can be observed in Figures 6.11 and 6.12, which occurred at strand stresses of 189 ksi (1301 MPa) and 196 ksi (1353 MPa), respectively. Note that, additional, continuously occurring losses in the post-tensioning forces are also observed in Figures 6.11 and 6.12 (possibly due to small amounts of nonlinear straining of the post-tensioning steel, additional seating of the anchor wedges, and/or nonlinear behavior in the beam/wall concrete/grout); however, these losses are small as compared with the losses due to the strand wire fractures.

As shown in Figure 6.8, each premature loss in the beam post-tensioning force results in an immediate reduction in the lateral resistance of the subassembly, a reduction in the self-centering capability of the structure upon unloading, and a reduction in the lateral stiffness during the subsequent loading cycles. This undesirable behavior was not anticipated during the experiment since the anchors were expected to develop at least 95% of the nominal design strength of the strands and 2.0% elongation (i.e., strain of 0.02) at fracture, as required by ACI 318 (ACI 2005). Since premature strand wire fractures were also observed in previous tests of similar unbonded post-tensioned coupled
wall subassemblies using steel beams (Weldon et al. 2006), measures were taken to prevent these fractures in the subsequent experiments as described later in this chapter.

Figure 6.11: Test 1 beam post-tensioning strand forces – (a) strand 1; (b) strand 2.

Figure 6.12: Test 1 $F_{LC}$-$\theta_{b,lb}$ behavior – (a) strand 1; (b) strand 2.
Figure 6.13: Test 1 beam post-tensioning force versus chord rotation behavior – (a) using beam displacements; (b) using load block displacements.

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6.2.5 Angle-to-Wall Connection Post-Tensioning Forces

The beam south end (i.e., reaction block end) angle-to-wall connection post-tensioning strand forces, $F_{LC3} - F_{LC6}$, measured using load cells LC3 – LC6, respectively, are shown in Figures 6.14 through 6.17. The target initial force for each connection strand is 20 kips (89 kN); whereas, the measured initial forces in the four strands are $F_{i,LC3} = 32.3$ kips (144 kN), $F_{i,LC4} = 17.2$ kips (76 kN), $F_{i,LC5} = 24.2$ kips (108 kN), and $F_{i,LC6} = 29.7$ kips (132 kN). A significant variation is observed in the initial connection strand forces since even a slight difference in the amount of anchor wedge seating has a large effect on the initial force (due to the short length of the strands).

Figures 6.18(a) and 6.18(b) show the total forces in the south top and south seat angle connection strands, respectively, plotted against the load block beam chord rotation, $\theta_{b,lb}$. The connection forces are normalized with the total design ultimate strength of the strands, $P_{abu} = \Sigma a_{ap} f_{apu}$, where $f_{apu} = 270$ ksi. The expected behavior of the strands is that as the structure is displaced and the angles are pulled in tension, the connection forces increase; and upon unloading, the connection forces return more or less back to the initial forces with possibly some losses occurring due to additional seating of the anchor wedges and any permanent deformations in the concrete (note that the nonlinear straining of the post-tensioning steel is prevented since the strands are left unbonded). However, this is not observed in Test 1, where the forces from LC3 – LC6 do not behave as expected. This could have been due to the malfunctioning of LC3 – LC6 under the non-uniform loads applied on the load cells during the prying deformations of the angles.
Figure 6.14: Test 1 south end top angle-to-wall connection strand forces – (a) east strand; (b) west strand.

Figure 6.15: Test 1 south end top angle-to-wall connection strand forces versus load block beam chord rotation – (a) $F_{LC3}$-$\theta_{b,lb}$; (b) $F_{LC4}$-$\theta_{b,lb}$. 
Figure 6.16: Test 1 south end seat angle-to-wall connection strand forces – (a) east strand; (b) west strand.

Figure 6.17: Test 1 south end seat angle-to-wall connection strand forces versus load block beam chord rotation – (a) $F_{LC5}$-$\theta_{b,lb}$; (b) $F_{LC6}$-$\theta_{b,lb}$. 
Figure 6.18: Test 1 south end total angle-to-wall connection strand forces versus load block beam chord rotation – (a) top connection; (b) seat connection.

6.2.6 Vertical Forces on Wall Test Region

Load cells LC7 – LC14 are used to measure the forces in the eight vertical bars applying axial compression forces to the wall test region of the reaction block and anchoring the block to the strong floor. The total vertical force, $F_{wt}$ is determined as described in Chapter 5, with the target initial total force ranging between 150 – 160 kips (667 – 712 kN). Figure 6.19(a) shows $F_{wt}$ for the duration of the test and Figure 6.19(b) plots $F_{wt}$ against the load block beam chord rotation, $\theta_{b,lb}$. The initial total force, $F_{wt,i}$ is 158 kips (703 kN), within the target force range. As the beam is rotated in the positive (i.e., clockwise) direction with the load block moving down, $F_{wt}$ decreases since the beam applies a downward force on the reaction block. Similarly, as the beam is rotated in the negative (i.e., counterclockwise) direction, $F_{wt}$ increases since the beam applies an upward force on the reaction block. Note that, as described in Chapter 5, the amount of
variation in $F_{wt}$ during the cyclic displacements of the beam is relatively small as compared with the expected variation of axial forces in the wall pier coupling regions of a multi-story coupled wall system. Upon unloading, $F_{wt}$ returns more or less to its initial value, with some amount of loss occurring possibly due to the loosening of the nuts on the bars and/or any permanent deformations in the reaction block concrete.

![Vertical Force on Wall Test Region](image1)

**Figure 6.19:** Test 1 vertical force on wall test region, $F_{wt}$ – (a) $F_{wr}$-test duration; (b) $F_{wr}$-$\theta_{b,lb}$.

### 6.2.7 Beam Vertical Displacements

The vertical displacements $\Delta DT9$ and $\Delta DT10$ at the south and north ends of the beam are measured using string pots DT9 and DT10, respectively. These displacements are used to calculate the beam chord rotation, $\theta_b$. As described in Chapter 5, as the subassembly is displaced, the transducer string undergoes a change of angle, which can be “adjusted” to give the vertical displacements in the y-direction. Figures 6.20 and 6.21 show the measured displacements $\Delta DT9$ and $\Delta DT10$, respectively, the corresponding adjusted y-displacements $\Delta DT9,y$ and $\Delta DT10,y$, respectively, and the differences between the
measured and adjusted displacements for the duration of the test. There is a downward drift in the data from DT9 starting at approximately 2.0% beam chord rotation, which was caused by the loosening of the ferrule insert as damage occurred at the south end of the beam.

It can be seen from Figures 6.20 and 6.21 that $\Delta_{DT9}$ and $\Delta_{DT10}$ are very close to $\Delta_{DT9,y}$ and $\Delta_{DT10,y}$, respectively, with the difference being less than 0.04 in. (1.0 mm). Figure 6.22 plots the percent difference between the measured and adjusted displacements versus the load block beam chord rotation, $\theta_{b,lb}$. The results indicate that most of the larger percent differences occur when the beam chord rotation is close to zero; however, these differences are not significant since the corresponding measurements are very small and are mostly outside the sensitivity of the transducers. It is observed that for negative rotations, DT9 displays larger percent errors than in the positive direction because the vertical displacements in the negative direction are smaller than the displacements in the positive direction (due to the loosening of the ferrule insert) as shown in Figure 6.23(a); therefore, the differences between the measured and the adjusted displacements result in larger percent errors. For DT10, the displacements in the negative and positive directions are more symmetric as shown in Figure 6.23(b).

Since the adjustments described in Chapter 5 require certain assumptions and approximations and since the differences between the measured and adjusted displacements remain small, these differences are ignored and the measurements from DT9 and DT10 are used as the vertical $y$-displacements of the beam throughout this dissertation. Figure 6.23 plots the measured data from DT9 and DT10 against the load block beam chord rotation, $\theta_{b,lb}$. 
Figure 6.20: Test 1 south end beam vertical displacements –
(a) measured, $\Delta_{DT9}$; (b) adjusted, $\Delta_{DT9,y}$; (c) difference, $\Delta_{DT9,y} - \Delta_{DT9}$. 
Figure 6.21: Test 1 north end beam vertical displacements –
(a) measured, $\Delta DT_{10}$; (b) adjusted, $\Delta DT_{10,y}$; (c) difference, $\Delta DT_{10,y} - \Delta DT_{10}$.

Figure 6.22: Test 1 percent difference between measured and adjusted displacements –
(a) south end, DT9; (b) north end, DT10.
6.2.8 Beam Chord Rotation

The beam chord rotation is defined as the relative vertical displacement of the beam ends divided by the beam length. As described in Chapter 5, the beam chord rotation is normally calculated using the vertical displacements of the beam from transducers DT9 and DT10 (i.e., $\Delta_{DT9}$ and $\Delta_{DT10}$, respectively); however, the vertical displacement of the load block centroid ($\Delta_{LB,y}$) is used to determine the beam chord rotation when DT9 and/or DT10 is not available (e.g., due to the loss of a sensor insert).

The beam chord rotation $\theta_b$ determined based on the $\Delta_{DT9}$ and $\Delta_{DT10}$ measurements in Test 1 is shown in Figure 6.24(a). As mentioned previously, the ferrule insert at the south end of the beam was lost due to the damage in the patched concrete; and thus, measurements from DT9 are not available after a beam chord rotation of 5.0%. For comparison, Figure 6.24(b) shows the load block beam chord rotation, $\theta_{b,lb}$ calculated using $\Delta_{LB,y}$, and Figure 6.25 shows the percent difference between $\theta_b$ and $\theta_{b,lb}$ plotted against $\theta_b$. It can be seen that there is a large percent difference for small beam chord...
rotations, with the difference dropping down to less than 10% for beam rotations up to 5.0%. Similar plots in Chapter 7 for Tests 3B and 4B demonstrate that the percent difference between $\theta_b$ and $\theta_{b,lb}$ remains small for beam chord rotations greater than 5.0% as well. Figure 6.26 plots the beam chord rotation, $\theta_b$ and the load block beam chord rotation, $\theta_{b,lb}$ against the beam chord rotation, $\theta_b$. The results show that the two chord rotations are approximately equal under positive loading, but have a small difference under negative loading with $\theta_{b,lb}$ being slightly smaller.

![Figure 6.24: Test 1 beam chord rotation – (a) $\theta_b$ from $\Delta DT9$ and $\Delta DT10$; (b) $\theta_{b,lb}$ from $\Delta LB,y$.](image-url)
Figure 6.25: Test 1 percent difference between $\theta_b$ and $\theta_{b,lb}$.

Figure 6.26: Test 1 difference between $\theta_b$ and $\theta_{b,lb}$. 
6.2.9 Local Beam Rotations

Local beam rotations were measured using two rotation transducers (inclinometers); one near the south end of the beam (RT1) and the other near the midspan (RT2). Figure 6.27 shows the rotation time history from RT2 (note that no data was collected from RT1 during Test 1), and Figure 7.25(b) compares $\theta_{RT2}$ with the beam chord rotation, $\theta_b$. The rotation measurements are positive when the load block is displaced in the downward direction (i.e., clockwise beam rotation). As a result of the bending deformations over the length of the beam, it may be expected that the midspan rotation, $\theta_{RT2}$ is larger than the chord rotation, $\theta_b$, and that the chord rotation is larger than the end rotation, $\theta_{RT1}$ [see Figure 6.28(c)]. However, since the nonlinear lateral displacements of the beam are primarily governed by the gap opening at the ends, the differences in the measured end, midspan, and chord rotations for the test beams are in general too small to make conclusive comparisons, and the different instruments utilized for the chord rotation measurements (using displacement transducers) and for the beam end and midspan rotation measurements (using rotation transducers) further make these comparisons difficult.
no data collected from RT1

(a)

Beam Rotation from Inclinometer near Midspan

(b)

Figure 6.27: Test 1 beam inclinometer rotations –
(a) near beam south end, $\theta_{RT1}$; (b) near beam midspan, $\theta_{RT2}$. 
no data collected from RT1

Figure 6.28: Test 1 percent difference between beam inclinometer rotations and beam chord rotations – (a) RT1; (b) RT2; (c) beam deflected shape.

6.2.10 Load Block Displacements and Rotations

String pots DT3 – DT5 are used to measure the vertical $y$-displacements and the horizontal $x$-displacement of the load block. Similar to the vertical beam displacements, as the load block is displaced, the strings of the load block displacement transducers undergo a change in angle; and thus, their measurements may need to be adjusted to give the $x$- and $y$-displacements of the load block as described in Chapter 5.

Figures 6.29 through 6.31 show the measured displacements $\Delta_{DT3}$, $\Delta_{DT4}$, and $\Delta_{DT5}$, respectively, the corresponding adjusted displacements $\Delta_{DT3,x}$, $\Delta_{DT4,y}$, and $\Delta_{DT5,y}$,
respectively, and the percent difference between the measured and adjusted
displacements. Note that, as described in Chapter 5, the negative $\Delta DT3$ and $\Delta DT3,x$
measurements indicate the movement of the load block in the north direction. It can be
seen that the difference between $\Delta DT3$ and $\Delta DT3,x$ is well over 10% for much of the
duration of the test; and thus, adjustments need to be applied to the measurements from
DT3. In comparison, the adjustments needed for the vertical displacement measurements
from DT4 and DT5 remain small throughout the test with the largest difference being
3.0%. Figure 6.32 shows the percent difference between the measured and adjusted
displacements for DT4 and DT5 plotted against the load block beam chord rotation, $\theta_{b,lb}$.
It can be seen that away from the origin, the maximum differences between the
unadjusted and adjusted measurements from DT4 and DT5 remain less than 0.3%.

The measurements from DT3 require larger adjustments than those from DT4 and
DT5 since the changes in the string angle for DT3, which occur due to the applied
vertical displacements of the load block, are much larger than the changes in the string
angles for DT4 and DT5, which occur due to the gap opening displacements at the beam
ends. In evaluating the results from Test 1, adjusted measurements are used for $\Delta DT3$;
however, the measurements for $\Delta DT4$ and $\Delta DT5$ are not adjusted. Figure 6.33 plots $\Delta DT4$,
$\Delta DT5$, and $\Delta DT3,x$ against the load block beam chord rotation, $\theta_{b,lb}$. It can be seen in Figures
6.33(a) and 6.33(b) that the vertical displacements of the load block at the north and
south ends are nearly the same.

Combining these displacements, the $x$-displacement, $y$-displacement, and rotation
of the load block centroid can be determined as described in Chapter 5 and shown in
Figures 6.34 and 6.35. Figure 6.34(a) plots the $y$-displacement versus the $x$-displacement
showing the path of the load block centroid during the test. As the subassembly is displaced under positive (i.e., clockwise) and negative (i.e., counterclockwise) rotations, the load block is pushed north in the $x$-direction (away from the reaction block) due to the gap opening at the beam ends. After each cycle, the load block returns to its initial position with minimal residual displacements. Figures 6.34(a) and 6.35(a) show that the $x$-direction displacements of the load block centroid are slightly smaller during the negative rotations of the subassembly as compared to the displacements during the positive rotations, which is possibly due to any unsymmetric loading and/or behavior of the structure. In the $y$-direction, the load block displaces symmetrically during the positive and negative rotations as shown in Figures 6.33(a), 6.33(b), 6.34(d), and 6.35(b). Finally, the rotation of the load block is shown to remain small throughout the duration of the test as plotted in Figures 6.34(b) and 6.35(c). The load block rotation remains below 0.004 radians indicating that the two hydraulic actuators moved near simultaneously.
Figure 6.29: Test 1 load block horizontal displacements – (a) measured, $\Delta DT_3$; (b) adjusted, $\Delta DT_{3,x}$; (c) percent difference.
Figure 6.30: Test 1 load block north end vertical displacements –
(a) measured, $\Delta DT4$; (b) adjusted, $\Delta DT4,y$; (c) percent difference.
Figure 6.31: Test 1 load block south end vertical displacements – (a) measured, $\Delta DT_5$; (b) adjusted, $\Delta DT_{5,y}$; (c) percent difference.

Figure 6.32: Test 1 percent difference between measured and adjusted displacements versus load block beam chord rotation – (a) DT4; (b) DT5.
Figure 6.33: Test 1 load block displacements versus load block beam chord rotation –
(a) $\Delta DT_4 - \theta_{b,lb}$; (b) $\Delta DT_5 - \theta_{b,lb}$; (c) $\Delta DT_{3,x} - \theta_{b,lb}$. 
Figure 6.34: Test 1 load block centroid displacements –
(a) x-y displacements; (b) rotation; (c) x-displacement; (d) y-displacement.
Figure 6.35: Test 1 load block centroid displacements versus load block beam chord rotation – (a) x-displacement-\(\theta_{b,lb}\); (b) y-displacement-\(\theta_{b,lb}\); (c) rotation-\(\theta_{b,lb}\).

6.2.11 Reaction Block Displacements and Rotations

String pots DT6 – DT8 are used to measure the vertical y-displacements and the horizontal x-displacement of the reaction block. Figure 6.26 shows the measurements from DT6 – DT8 for the duration of the test. Since the reaction block is tied to the strong floor, the measured displacements remain very small throughout the test. Due to these small displacements and the use of lead cables for each string pot, the change in angle that the string undergoes during testing is very small. Thus, it can be assumed that no
adjustments are needed for the displacements measured from the reaction block displacement transducers. Figure 6.37 plots the measurements from DT6 – DT8 against the load block beam chord rotation.

The $x$-displacement, $y$-displacement, and rotation of the reaction block centroid can be determined (see Chapter 5) as shown in Figure 6.38 and plotted against the load block beam chord rotation in Figure 6.39. It is concluded that the vertical displacements of the reaction block do not have a significant effect on the displacements of the test structure (e.g., the beam chord rotation), and the test results are evaluated with the reaction block displacements taken as zero (i.e., the measured displacements of the reaction block are ignored in investigating the response of the subassembly). Note that the horizontal displacements of the reaction block are significant when determining the total elongation of the post-tensioning tendon; and thus, are included in those calculations.
Figure 6.36: Test 1 reaction block displacements –
(a) $\Delta DT_6$; (b) $\Delta DT_7$; (c) $\Delta DT_8$. 
Figure 6.37: Test 1 reaction block displacements versus load block beam chord rotation – (a) $\Delta DT7-\theta_{b,lb}$; (b) $\Delta DT8-\theta_{b,lb}$; (c) $\Delta DT8-\theta_{b,lb}$. 
Figure 6.38: Test 1 reaction block centroid displacements – (a) $x$-$y$ displacements; (b) rotation; (c) $x$-displacement; (d) $y$-displacement.
Figure 6.39: Test 1 reaction block centroid displacements versus load block beam chord rotation – (a) $x$-displacements; (b) $y$-displacements; (c) rotation.
6.2.12 Contact Depth and Gap Opening at Beam-to-Wall Interfaces

The beam contact depth and gap opening displacements are measured at the beam-to-reaction-block interface using displacement transducers DT11 – DT13. As described in Chapter 5, these LVDTs rotate with the beam; and thus, their measurements may need to be adjusted to determine the gap opening displacements in the horizontal $x$-direction.

Figures 6.40 through 6.42 plot the measured displacements $\Delta_{DT11}$, $\Delta_{DT12}$, and $\Delta_{DT13}$, respectively, the corresponding adjusted $x$-displacements $\Delta_{DT11,x}$, $\Delta_{DT12,x}$, and $\Delta_{DT13,x}$, respectively, and the percent differences between the measured and adjusted displacements. The results indicate that the adjusted measurements are less than 0.5% different from the original measurements; and thus, $\Delta_{DT11}$, $\Delta_{DT12}$, and $\Delta_{DT13}$ can be taken as the displacements in the $x$-direction. Figure 6.43 plots the measured data from DT11, DT12, and DT13 against the load block beam chord rotation.

The maximum average concrete compressive strain in the beam-to-wall contact regions can be calculated by dividing the measured displacements from DT11 and DT13 with the gauge length (i.e., the distance from the LVDT ferrule insert in the beam to the reaction plate ferrule insert in the wall test region; see Chapter 5). For Test 1, the maximum average compressive strain is 0.0152. Note that this measurement includes compressive strain occurring in the fiber-reinforced patched concrete in the beam and wall test region of the reaction block, as well as the fiber-reinforced grout at the beam-to-wall interface.
Figure 6.40: Test 1 beam-to-reaction-block interface top LVDT displacements – (a) measured, $\Delta_{DT11}$; (b) adjusted, $\Delta_{DT11,x}$; (c) percent difference.
Figure 6.41: Test 1 beam-to-reaction-block interface middle LVDT displacements – (a) measured, $\Delta_{DT12}$; (b) adjusted, $\Delta_{DT12,x}$; (c) percent difference.
Figure 6.42: Test 1 beam-to-reaction-block interface bottom LVDT displacements – (a) measured, $\Delta_{DT13}$; (b) adjusted, $\Delta_{DT13,x}$; (c) percent difference.
Using the measured data, the contact depth and the largest (i.e., at the beam top and bottom) gap opening displacements at the beam-to-reaction-block interface can be determined following the procedures in Chapter 5. Figures 6.44(a) and 6.45(a) show the results based on the measured data from DT11 – DT13 (method 1); Figures 6.44(b) and 6.45(b) show the results based on the measured data from RT2, DT11, and DT13 (method 2); Figures 6.44(c) and 6.45(c) show the results based on the measured data from RT2 and DT12 (method 3); Figures 6.44(d) and 6.45(d) show the results based on the load block beam chord rotation, $\theta_{b,lb}$ and the measurements from DT12 (method 4); and
Figures 6.44(e) and 6.45(e) show the results based on the load block beam chord rotation, $\theta_{b,lb}$ and the measurements from DT11 and DT13 (method 5). Note that in Chapter 5, methods 2 and 3 use the data from RT1; however, because this measurement is not available for Test 1, the data from RT2 is used. Each $\circ$ marker in Figures 6.44 and 6.45 indicates the contact depth or gap opening displacement at the peak of a loading cycle up to a beam chord rotation of 6.4% (the LVDT ferrule inserts in the beam were lost during the third cycle to 6.4% rotation). Note that, as described in Chapter 5, the contact depth and gap opening results from methods 1, 3, and 4 are valid only when $\Delta_{DT12}$ is positive (i.e., the gap extends beyond the level of DT12 and the contact depth is less than $h_b/2$). Figures 7.40 and 7.42 show the results within the validity range of these methods.

Looking at Figure 6.44, it can be stated that the contact depth results obtained using the five methods are somewhat different (possibly due to the damage in the patched concrete, the differences in the measurements used in the different methods, and/or different amounts of loosening in the sensor inserts as damage occurred in the patched concrete, especially since the contact depth calculations are sensitive to these measurement differences) but show similar trends. There is a rapid reduction in the contact depth up to a beam chord rotation of about 2.0%. After this rotation, the contact depth remains relatively stable due to the nonlinear behavior of the concrete in compression and varies between 4.0 – 2.0 in. (102 – 51 mm) [about 30 – 15% of the beam depth]. In comparison, the gap opening results using the five methods in Figure 6.45 are reasonably similar and the increase in gap opening with the rotation of the beam is very close to linear. Note from Figure 6.7(a) that in Test 1, the deformations at the patched, south end of the beam occurred primarily in the deteriorating beam concrete.
instead of gap opening. Thus, the gap opening measurements in Figure 6.45 are more representative of the accumulated crack deformations in the patched concrete.

Figure 6.46 shows a continuous plot of the largest gap opening displacements determined from method 4 (using the load block beam chord rotation and DT12) against the load block beam chord rotation. Similarly, Figure 6.47(a) plots the beam contact depth from method 4 against the load block beam chord rotation as continuous data. Furthermore, Figures 6.47(b) and 6.47(c) show continuous plots of the contact depth during the 2.0% and 5.0% beam chord rotation cycles, respectively. Note that method 3 (using the measured data from RT1 and DT12) can also be used to obtain the continuous plots above; however, the measurements from RT1 were found to be not always reliable (especially at small rotations); and thus, method 4 is used instead.

It is observed that during the smaller displacement cycles of the subassembly (up through approximately 3.0% beam chord rotation), the beam is initially in full contact with the wall test region. As the system is loaded, a gap begins to open and a rapid reduction of the contact depth is observed in Figure 6.47(b), and upon removal of the load, the beam-to-wall interface returns back to full contact. During the larger displacement cycles of the subassembly, there is an accumulation of damage at the south end of the beam (especially in the patched concrete). This damage (e.g., cracks not fully closing) distorts the LVDT measurements near the zero position [i.e., at small $\theta_{b,lb}$ values, see Figure 6.47(c)]. As the beam is rotated, the cracks in the compression zone close further and the contact depth measurement approaches the behavior observed during the small rotation cycles. The gap opening and contact depth plots in Figures 6.46 and 6.47 could have been affected by the use of $\theta_{b,lb}$ (i.e., chord rotation) instead of $\theta_{RT1}$ (i.e., local rotation) to determine the
behavior of the south end of the beam. The use of the beam midheight transducer DT 12 instead of the extreme top or bottom transducer (DT11 or DT 13) may also have affected the results, especially the contact depth plots, in Figure 6.47 during small beam rotations, which are very sensitive to the measurements. Thus, the estimated contact depths at small beam rotations ($\theta_b < 0.25\%$) should be used with caution.

In addition to the LVDT measurements at the reaction block end (i.e., patched, south end) of the beam, the gap opening measurements at the load block end (i.e., unpatched, north end) of the beam were taken using a ruler. These ruler measurements are listed in Table 6.1 and shown using + markers in Figure 6.45. It can be seen that the LVDT gap opening measurements at the patched end (○ markers) are slightly smaller than but reasonably close to the ruler measurements at the unpatched end (+ markers). Thus, it can be stated that the accumulated crack deformations at the patched end of the beam are similar to the gap opening at the unpatched end.
Figure 6.44: Test 1 contact depth at beam-to-reaction-block interface – (a) method 1 using DT11, DT12, and DT13; (b) method 2 using RT2, DT11, and DT13; (c) method 3 using RT2 and DT12; (d) method 4 using $\theta_{b,lb}$ and DT12; (e) method 5 using $\theta_{b,lb}$, DT11, and DT13.
Figure 6.45: Test 1 gap opening at beam-to-reaction-block interface –
(a) method 1 using DT11, DT12, and DT13; (b) method 2 using RT2, DT11, and DT13;
(c) method 3 using RT2 and DT12; (d) method 4 using \( \theta_{b,lb} \) and DT12; (e) method 5
using \( \theta_{b,lb} \), DT11, and DT13.
Figure 6.46: Test 1 gap opening at beam-to-reaction-block interface using method 4.

Figure 6.47: Test 1 contact depth at beam-to-reaction-block interface using method 4 – (a) entire data; (b) 2.0% beam chord rotation cycle; (c) 5.0% beam chord rotation cycle.


### TABLE 6.1

**RULER MEASUREMENTS OF GAP OPENING AT NORTH BEAM END**

<table>
<thead>
<tr>
<th>Nominal Rotation (%)</th>
<th>Gap Opening, $\Delta_g$ [in. (mm)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>visible</td>
</tr>
<tr>
<td>0.35</td>
<td>paper thin</td>
</tr>
<tr>
<td>0.50</td>
<td>$&lt;0.125$ (3.175)</td>
</tr>
<tr>
<td>0.75</td>
<td>$&lt;0.125$ (3.175)</td>
</tr>
<tr>
<td>1.0</td>
<td>0.125 (3.175)</td>
</tr>
<tr>
<td>1.5</td>
<td>0.1875 (4.76)</td>
</tr>
<tr>
<td>2.0</td>
<td>0.21875 (5.56)</td>
</tr>
<tr>
<td>3.0</td>
<td>0.3125 (7.94)</td>
</tr>
<tr>
<td>4.0</td>
<td>0.5 (12.7)</td>
</tr>
<tr>
<td>5.0</td>
<td>0.625 (15.875)</td>
</tr>
<tr>
<td>6.4</td>
<td>0.75 (19.1)</td>
</tr>
<tr>
<td>8.0</td>
<td>0.8125 (20.64)</td>
</tr>
</tbody>
</table>

#### 6.2.13 Wall Test Region Local Concrete Deformations

The reaction block confined concrete deformations near the beam-to-wall interface of the wall test region are measured using displacement transducers DT14 and DT15. Figure 6.48 plots the time history results from the top (DT14) and bottom (DT15) transducers, and Figure 6.49 plots the measured data from DT14 and DT15 against the load block beam chord rotation. As expected, the concrete deformations are mostly compressive (negative) due to the compression stresses that are transferred through the contact region from post-tensioning.
From Figure 6.48, the maximum average concrete compressive strain in the wall test region can be calculated by dividing the measured displacements with the gauge length (i.e., the distance from the LVDT ferrule insert in the wall test region to the reaction plate; see Chapter 5). For Test 1, the maximum average concrete compressive strain is 0.0014, which is well below the expected unconfined (cover) concrete crushing strain of 0.004. This finding is in accordance with the visual observation that no concrete spalling occurred in the wall test region during the test.

![Reaction Block LVDT Displacement](image)

Figure 6.48: Test 1 wall test region concrete deformations – (a) $\Delta_{DT14}$; (b) $\Delta_{DT15}$. 
0.005 (0.13)

-0.38

-0.015 displacement, ΔDT14 [in. (mm)]

load block beam chord rotation, θb,lb (%)

01 0-10

0

(a)         (b)

Figure 6.49: Test 1 wall test region concrete deformations versus load block beam chord rotation – (a) ΔDT14-θb,lb; (b) ΔDT15-θb,lb.

6.2.14 Beam Looping Reinforcement Longitudinal Leg Strains

Figures 6.50 and 6.51 show the strain gauge measurements for the top and bottom horizontal legs of the east and west No. 6 mild steel looping reinforcing bars in the beam. The locations of these strain gauges can be found in Chapter 5. The initial beam post-tensioning force results in a small compressive strain in the bars at the beginning of the test. Upon lateral loading of the subassembly, the largest tensile strains occur, as expected, in the gauges closest to the angle-to-beam connection bolts [i.e., gauges 6(1)T-W, 6(1)B-E, and 6(1)B-W]. The measurements in the gauges away from the angle-to-beam connection decrease with distance from this critical location.

To provide a better understanding of the strain measurements in the horizontal legs of the beam looping reinforcement, the Δ and □ markers in Figure 6.50(b) for gauge 6(1)T-W correspond to positive and negative chord rotation peaks for the beam, respectively, and the ○ markers indicate zero rotation positions. To provide further insight into the results, Figures 6.52 and 6.53 show the strains plotted against the load
block beam chord rotation. In the positive (i.e., clockwise) rotation direction, the strains in the top bars increase in tension as the gap opens at the top south corner of the beam and the top angle is pulled in tension. In the negative (i.e., counterclockwise) direction, the strains in the bottom bars increase in tension and the top bars go into compression due to the closing of the gap.

The horizontal lines in Figures 6.50 through 6.53 show the yield strain of the longitudinal steel ($\varepsilon_{ly} = 0.00283$) from the material tests in Chapter 4. The maximum strain in gauge 6(1)B-E is only slightly above the steel yield strain; and thus, it is concluded that the amount of mild steel reinforcement used to transfer the angle forces into the beam is adequate. Note that the damage in the patched concrete at the south end of the beam resulted in larger than expected strains in the longitudinal reinforcing bars.

Figure 6.50: Test 1 beam looping reinforcement top longitudinal leg strains – (a) $\varepsilon_{6(1)T-E}$; (b) $\varepsilon_{6(1)T-W}$; (c) $\varepsilon_{6(2)T-E}$; (d) $\varepsilon_{6(2)T-W}$; (e) $\varepsilon_{6(3)T-E}$; (f) $\varepsilon_{6(3)T-W}$; (g) $\varepsilon_{6MT-E}$; (h) $\varepsilon_{6MT-W}$.
Figure 6.50 continued.
Figure 6.51: Test 1 beam looping reinforcement bottom longitudinal leg strains –
(a) $\varepsilon_{6(1)B-E}$; (b) $\varepsilon_{6(1)B-W}$; (c) $\varepsilon_{6(2)B-E}$; (d) $\varepsilon_{6(2)B-W}$; (e) $\varepsilon_{6(3)B-E}$; (f) $\varepsilon_{6(3)B-W}$; (g) $\varepsilon_{6MB-E}$; (h) $\varepsilon_{6MB-W}$.
Figure 6.51 continued.

Figure 6.52: Test 1 beam looping reinforcement top longitudinal leg strains versus load block beam chord rotation – (a) $\varepsilon_{6(1)T-E} - \theta_{b,lb}$; (b) $\varepsilon_{6(1)T-W} - \theta_{b,lb}$; (c) $\varepsilon_{6(2)T-E} - \theta_{b,lb}$; (d) $\varepsilon_{6(2)T-W} - \theta_{b,lb}$; (e) $\varepsilon_{6(3)T-E} - \theta_{b,lb}$; (f) $\varepsilon_{6(3)T-W} - \theta_{b,lb}$; (g) $\varepsilon_{6MT-E} - \theta_{b,lb}$; (h) $\varepsilon_{6MT-W} - \theta_{b,lb}$.
Figure 6.52 continued.
Figure 6.53: Test 1 beam looping reinforcement bottom longitudinal leg strains versus load block beam chord rotation – (a) $\varepsilon_{6(1)B-E}-\theta_{b,lb}$; (b) $\varepsilon_{6(1)B-W}-\theta_{b,lb}$; (c) $\varepsilon_{6(2)B-E}-\theta_{b,lb}$; (d) $\varepsilon_{6(2)B-W}-\theta_{b,lb}$; (e) $\varepsilon_{6(3)B-E}-\theta_{b,lb}$; (f) $\varepsilon_{6(3)B-W}-\theta_{b,lb}$; (g) $\varepsilon_{6MB-E}-\theta_{b,lb}$; (h) $\varepsilon_{6MB-W}-\theta_{b,lb}$.  

$\varepsilon_{fy} = 0.00283$
6.2.15 Beam Transverse Reinforcement Strains

Figure 6.54 shows the strain measurements from gauges 6SE(I)-E, 6SE(E)-E, 6SE(I)-W, and 6SE(E)-W placed on the transverse (i.e., vertical) legs of the east and west No. 6 looping reinforcing bars at the south end of the beam. The locations of these strain gauges can be found in Chapter 5. To give more insight into the measurements, Figure 6.55 plots the strain data against the load block beam chord rotation. It can be seen that the strain gauge readings remain well below the yield strain of the reinforcing steel ($\varepsilon_{y} = 0.00283$, see Chapter 4) throughout the experiment, demonstrating that the design of the transverse reinforcement at the beam ends is adequate. Since almost no concrete damage was observed at the unpatched (i.e., north) end of the beam and since the strains in the steel reinforcement remained small, it is concluded that the deterioration at the south end of the beam occurred due to the patched concrete. Note that the angle-to-beam connection bolts may also have acted as transverse reinforcement in the beam; however, this could not be confirmed from the test results since the connection bolts were not instrumented.
Similarly, Figure 6.56 shows the strain measurements from gauges MH-E and MH-W placed on the vertical legs of the No. 3 transverse hoop at the beam midspan. The location of these strain gauges can be found in Chapter 5. To give more insight into the measurements, Figure 6.57 plots the strain data against the load block beam chord rotation. As expected, the results indicate that the maximum strains in the midspan hoop remained well below the yield strain of the steel (\(\varepsilon_{h_y} = 0.00240\), see Chapter 4); and thus, the use of nominal transverse reinforcement within the span of the beam is adequate.

![Figure 6.54: Test 1 beam looping reinforcement vertical leg strains](image)

- (a) \(\varepsilon_{6SE(E)-E}\)
- (b) \(\varepsilon_{6SE(E)-W}\)
- (c) \(\varepsilon_{6SE(I)-E}\)
- (d) \(\varepsilon_{6SE(I)-W}\)
Figure 6.55: Test 1 beam looping reinforcement vertical leg strains versus load block beam chord rotation – (a) $\varepsilon_{6SE(E)-E} - \theta_{b,lb}$; (b) $\varepsilon_{6SE(E)-W} - \theta_{b,lb}$; (c) $\varepsilon_{6SE(I)-E} - \theta_{b,lb}$; (d) $\varepsilon_{6SE(I)-W} - \theta_{b,lb}$.

Figure 6.56: Test 1 beam midspan transverse hoop reinforcement strains – (a) $\varepsilon_{MH-E}$; (b) $\varepsilon_{MH-W}$.
6.2.16 Beam Confined Concrete Strains

Figures 6.58 and 6.59 show the measurements from the strain gauges placed on the No. 3 support bars inside the hoop confined concrete at the south end of the beam. The locations of these strain gauges can be found in Chapter 5. For further insight into the strain gauge readings, Figures 6.60 and 6.61 plot the strain data against the load block beam chord rotation. The results show that there is a small compressive strain at the beginning of the test due to the initial post-tensioning force. As the beam is rotated in the positive (i.e., clockwise) direction, the compression strains in the bottom bars increase due to the transfer of the contact stresses through the bottom corner of the beam. In the opposite (i.e., counterclockwise) direction, the strains in the top and bottom bars reverse due to the reversal of the load.

For most of the gauges, the strains in the support bars remain mostly compressive. This is expected since the bars do not cross the beam-to-reaction-block interface where the tensile deformations concentrate due to gap opening. As shown in Figure 6.7(a),
under large rotations of the beam, the tensile deformations at the south end of the beam occurred primarily in the deteriorating patch concrete instead of gap opening. There was also significant crushing of the patched concrete in compression. Note that while the strain measurement $\varepsilon_{3BHT-(2)}$ exceeds the yield strain $\varepsilon_{hy} = 0.00240$ of the No. 3 support bar in tension [see horizontal lines in Figures 6.59(d) and 6.61(d)] and the measurement $\varepsilon_{3THB-(2)}$ exceeds the expected crushing strain $\varepsilon_{cu} = 0.004$ of the unconfined concrete in compression, the measurement from the other strain gauges do not exceed these limits. Furthermore, all of the strain measurements remain significantly below the expected crushing of the confined concrete. Thus, it can be stated that these measurements are not representative of the amount of damage observed at the beam ends, possibly due to the deterioration of bond between the steel and concrete.
Figure 6.58: Test 1 No. 3 top hoop support bar strains – (a) $\varepsilon_{3THT-(1)}$; (b) $\varepsilon_{3THT-(1)}$; (c) $\varepsilon_{3THT-(2)}$; (d) $\varepsilon_{3THB-(2)}$. 
Figure 6.59: Test 1 No. 3 bottom hoop support bar strains –
(a) $\varepsilon_{3BHB-(1)}$; (b) $\varepsilon_{3BHT-(1)}$; (c) $\varepsilon_{3BHB-(2)}$; (d) $\varepsilon_{3BHT-(2)}$. 
Figure 6.60: Test 1 No. 3 top hoop support bar strains versus load block beam chord rotation – (a) $\varepsilon_{3THT-(1)} - \theta_{b,lb}$; (b) $\varepsilon_{3THB-(1)} - \theta_{b,lb}$; (c) $\varepsilon_{3THT-(2)} - \theta_{b,lb}$; (d) $\varepsilon_{3THB-(2)} - \theta_{b,lb}$. 
Figures 6.61 and 6.63 show the measurements from the strain gauges placed on the vertical legs of the bottom layer No. 3 confinement hoops at the south end of the beam. The locations of these strain gauges can be found in Chapter 5. There was no confinement hoop bar fracture during the test even though a significant amount of compression damage was observed in the confined concrete starting at approximately 5.0% rotation. To give more insight into the hoop strain measurements, Figures 6.64 and

6.2.17 Beam End Confinement Hoop Strains

Figures 6.62 and 6.63 show the measurements from the strain gauges placed on the vertical legs of the bottom layer No. 3 confinement hoops at the south end of the beam. The locations of these strain gauges can be found in Chapter 5. There was no confinement hoop bar fracture during the test even though a significant amount of compression damage was observed in the confined concrete starting at approximately 5.0% rotation. To give more insight into the hoop strain measurements, Figures 6.64 and
6.65 plot the strain data against the load block beam chord rotation. The angle-to-beam connection bolt forces result in a small compressive strain in the vertical hoop steel at the beginning of the test. Throughout much of the test, the measured confinement hoop strains remain small. Strain gauges 2HB-E, 3HB-E, and 3HB-W experience a large increase in compressive strain near the end of the test, which is most likely due to the damage at the south end of the beam. Figure 6.66 shows the east and west sides of the south end of the beam with significant concrete damage and exposed confinement hoops. The significant loss of cover concrete around the hoops and the presence of high compressive stresses in the vertical direction (which develop due to the external confining effect of the angle horizontal legs, see Figure 6.67) caused the vertical legs of the hoops to bend outward, resulting in the strain gauge measurements to be distorted. Note that the strain gauges were placed on the inside of the vertical legs, leading to the development of compression strains due to the outward bending of the hoop reinforcement.
Figure 6.62: Test 1 beam end confinement hoop east leg strains – (a) $\varepsilon_{1HB-E}$; (b) $\varepsilon_{2HB-E}$; (c) $\varepsilon_{3HB-E}$; (d) $\varepsilon_{4HB-E}$. 
Figure 6.63: Test 1 beam end confinement hoop west leg strains — (a) $\varepsilon_{1HB-W}$; (b) $\varepsilon_{2HB-W}$; (c) $\varepsilon_{3HB-W}$; (d) $\varepsilon_{4HB-W}$.
Figure 6.64: Test 1 beam end confinement hoop east leg strains versus load block beam chord rotation – (a) $\varepsilon_{1HB-E} - \theta_{b,lb}$; (b) $\varepsilon_{2HB-E} - \theta_{b,lb}$; (c) $\varepsilon_{3HB-E} - \theta_{b,lb}$; (d) $\varepsilon_{4HB-E} - \theta_{b,lb}$.
Figure 6.65: Test 1 beam end confinement hoop west leg strains versus load block beam chord rotation – (a) $\varepsilon_{1HB-W}-\theta_{b,lb}$; (b) $\varepsilon_{2HB-W}-\theta_{b,lb}$; (c) $\varepsilon_{3HB-W}-\theta_{b,lb}$; (d) $\varepsilon_{4HB-W}-\theta_{b,lb}$. 
Figure 6.66: Photograph of damage on south end of beam – (a) east side; (b) west side.
6.2.18 Wall Test Region Confined Concrete Strains

Figures 6.68 and 6.69 show the measurements from the strain gauges placed on the No. 3 corner bars inside the hoop confined concrete of the wall test region of the reaction block below the main post-tensioning duct. The locations of these strain gauges can be found in Chapter 5. There is a small compressive strain at the beginning of the test
due to the initial post-tensioning force. To give more insight into the measurements, Figures 6.70 and 6.71 plot the strain data against the load block beam chord rotation. It can be seen that for positive rotations, compressive strains develop in the confined concrete as the contact stresses at the south bottom corner of the beam are transferred into the reaction block concrete. In the negative rotation direction, small tensile strains develop in the same region, possibly due to the shift of the contact zone to the top corner of the beam. Strain gauge CBM1W shows tensile strains throughout most of the duration of the test; however, it displays the same trends in Figure 6.71 as the other strain gauges. It is possible that the location of this strain gauge and its orientation may have caused it not to be subjected to any compressive strains. Looking at the average compressive strains from CBM1E and CBM1W in Figure 6.72, a more representative behavior of the wall test region of the reaction block can be seen. The results shows the development of compressive stresses below the main post-tensioning duct in the wall test region of the reaction block below under positive (i.e., clockwise) rotations and tensile stresses in the same region of the reaction block under negative (i.e., counterclockwise) rotations. Both the compressive and tensile strains remain very small throughout the duration of the test, confirming the observed behavior of the reaction block with no cracking or spalling throughout the test.
Figure 6.68: Test 1 reaction block corner bar strains –
(a) $\varepsilon_{\text{CBB1E}}$; (b) $\varepsilon_{\text{CBB1W}}$; (c) $\varepsilon_{\text{CBB2E}}$; (d) $\varepsilon_{\text{CBB2W}}$; (e) $\varepsilon_{\text{CBB3E}}$; (f) $\varepsilon_{\text{CBB3W}}$. 
Figure 6.69: Test 1 reaction block corner bar strains – (a) $\varepsilon_{CBM1E}$; (b) $\varepsilon_{CBM1W}$; (c) $\varepsilon_{CBM3E}$; (d) $\varepsilon_{CBM3W}$. 
Figure 6.70: Test 1 reaction block corner bar strains versus load block beam chord rotation
   – (a) $\varepsilon_{CBB1E}-\theta_{b,lb}$; (b) $\varepsilon_{CBB1W}-\theta_{b,lb}$; (c) $\varepsilon_{CBB2E}-\theta_{b,lb}$; (d) $\varepsilon_{CBB2W}-\theta_{b,lb}$; (e) $\varepsilon_{CBB3E}-\theta_{b,lb}$; (f) $\varepsilon_{CBB3W}-\theta_{b,lb}$.
Figure 6.71: Test 1 reaction block corner bar strains versus load block beam chord rotation – (a) $\varepsilon_{CBM1E}$-$\theta_{b,lb}$; (b) $\varepsilon_{CBM1W}$-$\theta_{b,lb}$; (c) $\varepsilon_{CBM3E}$-$\theta_{b,lb}$; (d) $\varepsilon_{CBM3W}$-$\theta_{b,lb}$.

Figure 6.72: Test 1 reaction block corner bar average strains – (a) test duration; (b) versus load block beam chord rotation.
6.2.19 Wall Test Region Confinement Hoop Strains

Figure 6.73 shows the measurements from the strain gauges placed on the first layer No. 3 confinement hoops in the wall test region of the reaction block below the main post-tensioning duct. The locations of these strain gauges can be found in Chapter 5. To give more insight into the measurements, Figure 6.74 plots the strain data against the load block beam chord rotation. The strains remain very small throughout the duration of the test. The two top hoops, corresponding to strain gauges 1THM and 1MHM, are nearest the contact region that develops at the beam-to-wall interface. The bottom hoop strain gauge, 1BHE, goes into compression during the test. This is due to the placement of the strain gauge on the vertical leg of the confinement hoop, resulting in compression strains from the applied vertical tie-down forces in the wall test region of the reaction block. The other two strain gauges, 1THM and 1MHM, are placed on the horizontal legs of the hoops; and thus, are not affected by the vertical forces in the wall test region.
Figure 6.73: Test 1 reaction block hoop strains – (a) $\varepsilon_{1THM}$; (b) $\varepsilon_{1MHM}$; (c) $\varepsilon_{1BHE}$. 
6.2.20 Crack Patterns

The crack propagation from testing is marked on the beam during each cycle of displacement (e.g., see Figures 6.4, 6.5 and 6.6). In addition, the crack propagation is recorded by approximately copying the crack patterns manually onto paper. Figure 6.75 shows the hand-drawn crack patterns recorded for Test 1 at the end of the last cycle from selected displacement increments (see Appendix F for crack sheets).
Figure 6.75: Test 1 crack patterns – (a) $\theta_b = 0.35\%$; (b) $\theta_b = 0.5\%$; (c) $\theta_b = 0.75\%$; (d) $\theta_b = 1.0\%$; (e) $\theta_b = 1.5\%$; (f) $\theta_b = 2.0\%$; (g) $\theta_b = 3.0\%$. 
6.3 Test 2

The beam used in Test 2 (i.e., Beam 2) has the following properties: (1) beam depth, \(h_b = 14\) in. (356 mm); (2) beam width, \(b_b = 7.5\) in. (191 mm); (3) mild steel reinforcement of two No. 6 bars looping around the beam perimeter along its length; (4) No. 3 full-depth rectangular hoops [6.125 in. by 12.675 in. (156 mm by 322 mm)] placed at a nominal 7.0 in. (178 mm) spacing to provide transverse reinforcement in the beam midspan region; (5) No. 3 partial-depth rectangular hoops [6.125 in. by 4.375 in. (156 mm by 111 mm)] placed at a 1.5 in. (38 mm) spacing to provide concrete confinement at the beam ends; (6) a beam post-tensioning tendon comprised of four 0.6 in. (15 mm) nominal diameter high-strength strands with a total area of \(A_{bp} = 0.868\) in.\(^2\) (560 mm\(^2\)); (7) average initial beam post-tensioning strand stress of \(f_{hpi} = 0.36f_{bpu}\), where \(f_{bpu} = 270\) ksi (1862 MPa) is the design maximum strength of the post-tensioning steel; (8) total initial beam post-tensioning force of \(P_{bi} = 84.2\) kips (374 kN); (9) initial beam concrete nominal axial stress (based on actual cross-sectional area with beam post-tensioning duct removed) of \(f_{bci} = 0.82\) ksi (5.7 MPa); and (10) two top and two seat angles (L8x8x1/2) with length, \(l_a = b_b = 7.5\) in. (191 mm).

The primary parameter differences of Test 2 from Test 1 are: (1) increased beam post-tensioning area; (2) reduced initial beam post-tensioning strand stress; (3) increased initial beam concrete nominal axial stress; and (4) reduced height of the beam-to-wall interface grout column. The test subassembly did not have any patched concrete due to the use of a new beam and a new reaction block that were properly cast. In addition, a slightly different displacement loading history was used in Test 2 as described in Chapter
3. Furthermore, an extra barrel was used between each barrel/wedge anchor and the anchor plate to help prevent premature fracture of the post-tensioning strands. This is discussed in more detail in Section 6.3.4.

6.3.1 Test Photographs

Photographs of the original and displaced subassembly configurations from Test 2 are shown in Figures 6.76 and 6.77. Figures 6.76(a) through 6.76(f) show overall subassembly photographs as follows: (a) pre-test undisplaced position; (b) displaced to \( \theta_b = 3.33\% \); (c) displaced to \( \theta_b = -3.33\% \); (d) displaced to \( \theta_b = 6.4\% \); (e) displaced to \( \theta_b = -6.4\% \); and (f) final post-test undisplaced position. Similarly, Figures 6.77(a) through 6.77(f) show close-up photographs of the south end of the beam at the beam-to-reaction-block interface. The accumulation of damage at the south end of the beam is shown in more detail in Figure 6.78.

The wall test region of the reaction block experienced a small amount of cover concrete spalling near the beam centerline during the test; however, no additional damage was observed in the block. Spalling of the cover concrete at the beam ends initiated at a beam chord rotation of \( \theta_b = 1.5\% \); however, the spalling was relatively small up to a rotation of \( \theta_b = 3.33\% \). As the beam chord rotation increased beyond 3.33\%, the beam ends suffered a significant amount of damage, which played an important role in the behavior of the subassembly as described later.

The angle-to-wall connections performed well with no yielding in the connection strands and no damage to the wall concrete. A small amount of slip in the angle-to-beam connections between the angles and the beam was observed during the test, most likely
due to the loss of force in the angle-to-beam connection bolts caused by the deterioration of the concrete at the beam ends. Slip between the coupling beam and the walls did not occur demonstrating that the friction resistance due to the post-tensioning force provided adequate vertical support to the beam together with resistance from the top and seat angles. Unlike Test 1, no premature wire fractures were observed in the post-tensioning strands in Test 2.

The fiber-reinforced grout columns at the beam-to-wall interfaces in Test 2 were terminated approximately 0.25 in. (6 mm) short of the full beam depth as shown in Figure 6.79(a). This prevented the angle heels from coming into contact with the grout columns under large rotations; and thus, prevented the buckling of the grout columns, which was observed in Test 1. Under smaller displacements of the coupling beam up to about 3.33% rotation, gap opening occurred between the grout and the faces of the load and reaction blocks. Upon further displacements of the structure, gap opening continued to form until the end of the test; however, it was observed that gaps formed on both sides of the grout column [Figure 6.79(b)]. While this is not the desired mode of behavior for the structure as described in Chapter 3, it did not change the behavior of the system or cause any problems with the performance of the grout. No significant crushing of the grout at the beam-to-wall interfaces was observed throughout the test.
Figure 6.76: Test 2 overall photographs – (a) pre-test undisplaced position; (b) $\theta_b = 3.33\%$; (c) $\theta_b = -3.33\%$; (d) $\theta_b = 6.4\%$; (e) $\theta_b = -6.4\%$; (f) final post-test undisplaced position.
Figure 6.77: Test 2 beam south end photographs – (a) pre-test undisplaced position; (b) $\theta_b = 3.33\%$; (c) $\theta_b = -3.33\%$; (d) $\theta_b = 6.4\%$; (e) $\theta_b = -6.4\%$; (f) final post-test undisplaced position.
Figure 6.78: Test 2 beam south end damage propagation – positive and negative rotations.

Figure 6.79: Test 2 grout and gap opening behavior at south end – (a) $\theta_b = -0.5\%$; (b) $\theta_b = -5.0\%$. 
6.3.2 Beam Shear Force versus Chord Rotation Behavior

Figure 6.80(a) shows the hysteretic coupling beam shear force, $V_b$, versus chord rotation, $\theta_b$, behavior from Test 2, where $V_b$ and $\theta_b$ are calculated from Equations 5.5 and 5.35, respectively. Due to instrument malfunction, data was not collected from DT9; and thus, the beam chord rotation, $\theta_b$ in Figure 6.80(a) is calculated with $\Delta_{DT9}$ taken as zero. Note that $\Delta_{DT9} = 0$ is a reasonable assumption since DT9 is at the south end of the beam close to the reaction block tied to the strong floor. For comparison, Figure 6.80(b) plots the $V_b$-$\theta_{b,lb}$ behavior, where $\theta_{b,lb}$ is the beam chord rotation determined from the vertical ($y$-direction) displacement of the load block centroid using Equation 5.36. From Test 1, it was determined that the load block beam chord rotation, $\theta_{b,lb}$ is a more accurate measure of the actual beam chord rotation than $\theta_b$ calculated using Equation 5.35 with $\Delta_{DT9}$ taken as zero. Therefore, the results for Test 2 are presented using $\theta_{b,lb}$ as the beam chord rotation. This is discussed further in Section 6.3.8.

As shown in Figure 6.80, the structure was able to sustain three cycles at 6.4% rotation with approximately 9.8% loss in shear resistance. The test was stopped at this point due to the significant amount of damage that the beam had sustained (see Figures 6.76, 6.77, 6.78, and 6.81). Looking at the hysteresis loops, it can be seen that the specimen was able to dissipate a considerable amount of energy. Most of this energy dissipation occurred due to the yielding of the top and seat angles. The post-tensioning tendon did not have any wire fracture during the test due to the reduced initial stress in the post-tensioning strands and the of additional anchors to reduce strand “kinking” inside the anchors (see Section 6.3.4). The beam had a sufficient amount of restoring force to yield the tension angles back in compression and close the gaps at the beam ends.
upon unloading, resulting in a large self-centering capability; however, the increase in the initial beam concrete stress led to higher concrete compressive stresses resulting in a larger amount of damage to the beam as compared with Test 1.
Figure 6.80: Test 2 coupling beam shear force versus chord rotation behavior – (a) using beam displacements; (b) using load block displacements.
Figure 6.81: Test 2 damage to beam ends (upon angle removal after completion of test) – (a) south end; (b) north end.

6.3.3 Beam End Moment Force versus Chord Rotation Behavior

Figure 6.82 shows the hysteretic coupling beam end moment, $M_b$ versus chord rotation, $\theta_{b,lb}$ behavior from Test 2, where $M_b$ is calculated from Equation 5.6 as described in Chapter 5 and $\theta_{b,lb}$ is determined using the load block centroid displacements as described in the previous section. Since the beam end moment is calculated from the beam shear force, the results shown in Figure 6.82 are directly related to the results in Figure 6.80, and thus, no further discussion is provided herein.
6.3.4 Beam Post-Tensioning Forces

The coupling beam post-tensioning forces from the test are measured using load cells LC15 – LC18 mounted at the dead ends of the four strands (see Chapter 5). The forces from the four load cells, $F_{LC15}$, $F_{LC16}$, $F_{LC17}$, and $F_{LC18}$, are shown in Figure 6.83, and are plotted against the load block beam chord rotation, $\theta_{b,lb}$ in Figure 6.84. Figure 6.85 shows the total beam post-tensioning tendon force, $P_{bp}$ (sum of the forces in the four strands) normalized with the total design ultimate strength of the tendon, $\Sigma a_{bp}f_{pu}$. The total initial beam post-tensioning tendon force is equal to 84.2 kips (374 kN), resulting in an initial beam nominal concrete axial stress of $f_{bci} = 0.82$ ksi (5.7 MPa).
As the structure is displaced and gap opening occurs at the beam ends, the post-tensioning strands elongate and the post-tensioning forces increase. Since the strands are unbonded over the entire length of the subassembly, the nonlinear straining of the post-tensioning steel is significantly delayed with only small amounts of post-tensioning losses occurring in Figures 6.83 and 6.84 (possibly due to small amounts of nonlinear straining of the post-tensioning steel, additional seating of the anchor wedges, and/or nonlinear behavior in the beam/wall concrete).

Note that premature wire fractures of the post-tensioning strands were observed in Test 1, resulting in sudden and unexpected drops in the post-tensioning force. To prevent similar fractures from happening in Test 2, the initial post-tensioning stress in each strand was reduced. Furthermore, as shown in Figure 6.86, an additional anchor barrel (with no wedges) was placed between each anchor and anchor bearing plate (or load cell on dead end) to help reduce the amount of strand “kinking” inside the anchor as the structure was displaced laterally. As the subassembly is cycled, the strand “kinks” at the anchor enabling the wedge teeth to dig into the individual wires creating a weak point and leading to wire fracture. The additional anchor barrel helps to limit the amount of kinking inside the wedges. Note that the orientation of the additional barrel is important as shown in Figure 6.86. The above measures taken in this dissertation were successful in preventing premature wire fractures in the post-tensioning strands; however, more research is needed in this area as described in Chapter 11 and investigated in Walsh and Kurama (2009).
Figure 6.83: Test 2 beam post-tensioning strand forces – (a) strand 1; (b) strand 2; (c) strand 3; (d) strand 4.
Figure 6.84: Test 2 $F_{LC-\theta_{b,lb}}$ behavior –
(a) strand 1; (b) strand 2; (c) strand 3; (d) strand 4.
Figure 6.85: Test 2 beam post-tensioning force versus chord rotation behavior –
(a) using beam displacements; (b) using load block displacements.
Figure 6.86: Use of additional anchor barrel to prevent strand wire fracture.

6.3.5 Angle-to-Wall Connection Post-Tensioning Forces

The beam south end (i.e., reaction block end) angle-to-wall connection post-tensioning strand forces, $F_{LC3} - F_{LC6}$, measured using load cells LC3 – LC6, respectively, are shown in Figures 6.87 through 6.90. The target initial force for each connection strand is 20 kips (89 kN); whereas, the measured initial forces in the four strands are $F_{i,LC3} = 32.0$ kips (142 kN), $F_{i,LC4} = 37.0$ kips (165 kN), $F_{i,LC5} = 30.4$ kips (135 kN), and $F_{i,LC6} = 27.0$ kips (120 kN). A significant variation is observed in the initial connection strand forces since even a slight difference in the amount of anchor wedge seating has a large effect on the initial force (due to the short length of the strands).

Figures 6.91(a) and 6.91(b) show the total forces in the south top and south seat angle connection strands, respectively, plotted against the load block beam chord rotation, $\theta_{b,lb}$. The connection forces are normalized with the total design ultimate
strength of the strands, $P_{abu} = \Sigma a_{ap} f_{apu}$, where $f_{apu} = 270$ ksi. The expected behavior of the strands is that as the structure is displaced and the angles are pulled in tension, the connection forces increase; and upon unloading, the connection forces return more or less back to the initial forces with possibly some losses occurring due to additional seating of the anchor wedges and any permanent deformations in the concrete (note that the nonlinear straining of the post-tensioning steel is prevented since the strands are left unbonded). These trends are seen on the top angle load cells (LC3 and LC4) in Figure 6.91(a); however, the forces from LC5 and LC6 [Figure 6.91(b)] do not behave as expected. This could have been due to the malfunctioning of LC5 and LC6 under the non-uniform loads applied on the load cells during the prying deformations of the angles.
Figure 6.87: Test 2 south end top angle-to-wall connection strand forces – (a) east strand; (b) west strand.

Figure 6.88: Test 2 south end top angle-to-wall connection strand forces versus load block beam chord rotation – (a) $F_{LC3}$-$\theta_{b,lb}$; (b) $F_{LC4}$-$\theta_{b,lb}$.
Figure 6.89: Test 2 south end seat angle-to-wall connection strand forces – (a) east strand; (b) west strand.

Figure 6.90: Test 2 south end seat angle-to-wall connection strand forces versus load block beam chord rotation – (a) $F_{LC5} - \theta_{b,lb}$; (b) $F_{LC6} - \theta_{b,lb}$.
6.3.6 Vertical Forces on Wall Test Region

Load cells LC7 – LC14 are used to measure the forces in the eight vertical bars applying axial compression forces to the wall test region of the reaction block and anchoring the block to the strong floor. The total vertical force, $F_{wt}$, is determined as described in Chapter 5, with the target initial total force ranging between 150 – 160 kips (667 – 712 kN). Figure 6.92(a) shows $F_{wt}$ for the duration of the test and Figure 6.92(b) plots $F_{wt}$ against the load block beam chord rotation, $\theta_{lb,lb}$. The initial total force, $F_{wt,i}$, is 145 kips (645 kN), slightly under the target force range. As the beam is rotated in the positive (i.e., clockwise) direction with the load block moving down, $F_{wt}$ decreases since the beam applies a downward force on the reaction block. Similarly, as the beam is rotated in the negative (i.e., counterclockwise) direction, $F_{wt}$ increases since the beam applies an upward force on the reaction block. Note that, as described in Chapter 5, the

Figure 6.91: Test 2 south end total angle-to-wall connection strand forces versus load block beam chord rotation – (a) top connection; (b) seat connection.

(a)  
(b)
amount of variation in $F_{wt}$ during the cyclic displacements of the beam is relatively small as compared with the expected variation of axial forces in the wall pier coupling regions of a multi-story coupled wall system. Upon unloading, $F_{wt}$ returns more or less to its initial value, with some amount of loss occurring possibly due to the loosening of the nuts on the bars and/or any permanent deformations in the reaction block concrete.

![Figure 6.92: Test 2 vertical force on wall test region, $F_{wt}$ — (a) $F_{wt}$-test duration; (b) $F_{wt}$-$\theta_{b,lb}$.

6.3.7 Beam Vertical Displacements

The vertical displacements $\Delta_{DT9}$ and $\Delta_{DT10}$ at the south and north ends of the beam are measured using string pots DT9 and DT10, respectively. These displacements are used to calculate the beam chord rotation, $\theta_b$. As described in Chapter 5, as the subassembly is displaced, the transducer string undergoes a change of angle, which can be “adjusted” to give the vertical displacements in the $y$-direction. Figures 6.93 and 6.94 show the measured displacements $\Delta_{DT9}$ and $\Delta_{DT10}$, respectively, the corresponding
adjusted \( y \)-displacements \( \Delta_{DT9,y} \) and \( \Delta_{DT10,y} \), respectively, and the differences between the measured and adjusted displacements for the duration of the test.

Unfortunately, no data was collected from DT9; however, it can be seen from Figure 6.93 that \( \Delta_{DT10} \) is very close to \( \Delta_{DT10,y} \), with the difference being less than 0.02 in. (0.5 mm). Figure 6.95 plots the percent difference between the measured and adjusted displacements versus the load block beam chord rotation, \( \theta_{b,lb} \). The results indicate that the largest percent differences occur when the beam chord rotation is close to zero; however, these differences are not significant since the corresponding measurements are very small and are mostly outside the sensitivity of the transducers. Since the adjustments described in Chapter 5 require certain assumptions and approximations and since the percent differences between the measured and adjusted displacements remain small, these differences are ignored and the measurements from DT10 are used as the vertical \( y \)-displacements at the north end of the beam throughout this dissertation. Figure 6.96 plots the measured data from DT10 against the load block beam chord rotation, \( \theta_{b,lb} \).

![South Beam Vertical Displacement](image)

Figure 6.93: Test 2 south end beam vertical displacement.
Figure 6.94: Test 2 north end beam vertical displacements – (a) measured, $\Delta_{DT10}$; (b) adjusted, $\Delta_{DT10,y}$; (c) difference, $\Delta_{DT10,y} - \Delta_{DT10}$.

Figure 6.95: Test 2 percent difference between measured and adjusted displacements – (a) south end, DT9; (b) north end, DT10.
6.3.8 Beam Chord Rotation

The beam chord rotation is defined as the relative vertical displacement of the beam ends divided by the beam length. As described in Chapter 5, the beam chord rotation is normally calculated using the vertical displacements of the beam from transducers DT9 and DT10 (i.e., $\Delta_{DT9}$ and $\Delta_{DT10}$, respectively); however, the vertical displacement of the load block centroid ($\Delta_{LB}$) is used to determine the beam chord rotation when DT9 and/or DT10 is not available (e.g., due to the malfunction of a sensor).

As mentioned previously, DT9 did not work properly during the test and no data was collected from it. The beam chord rotation $\theta_b$ determined using $\Delta_{DT10}$ with $\Delta_{DT9}$ taken as zero throughout Test 2 is shown in Figure 6.97(a). Note that $\Delta_{DT9} = 0$ is a reasonable assumption since DT9 is at the south end of the beam, close to the reaction block tied to the strong floor. However, from Test 1, it was concluded that the load block beam chord
rotation is a more accurate measure of the actual beam chord rotation than the rotation calculated using $\Delta DT_{10}$ with $\Delta DT_{9}$ taken as zero.

For comparison, Figure 6.97(b) shows the load block beam chord rotation, $\theta_{b,lb}$ calculated using $\Delta LB_{y}$, and Figure 6.98 shows the percent difference between $\theta_b$ and $\theta_{b,lb}$ plotted against $\theta_b$. It can be seen that there is a large percent difference for small beam chord rotations, with the difference dropping down to less than 10% at larger beam rotations. Figure 6.99 plots the beam chord rotation, $\theta_b$ and the load block beam chord rotation, $\theta_{b,lb}$ against the beam chord rotation, $\theta_b$. The results show that the two chord rotations are approximately equal under negative loading, but have a small difference under positive loading with $\theta_{b,lb}$ being slightly smaller.

![Beam Chord Rotation from Beam Displacements](a)

![Beam Chord Rotation from Load Block Displacements](b)

Figure 6.97: Test 2 beam chord rotation – (a) $\theta_b$ from $\Delta DT_{10}$ with $\Delta DT_{9}$ taken as zero; (b) $\theta_{b,lb}$ from $\Delta LB_{y}$. 325
Figure 6.98: Test 2 percent difference between $\theta_b$ and $\theta_{b,lb}$.

Figure 6.99: Test 2 difference between $\theta_b$ and $\theta_{b,lb}$. 
6.3.9 Local Beam Rotations

Local beam rotations were measured using two rotation transducers (inclinometers); one near the midspan of the beam (RT1) and the other near the south end (RT2). Figure 6.100 shows the rotation time history from RT2 (note that no data was collected from RT1 during Test 2). The rotation measurements are positive when the load block is displaced in the downward direction (i.e., clockwise beam rotation). When compared to the beam chord rotations calculated from the load block displacements as shown in Figure 6.101, at larger rotations, the measurements from RT2 are larger, with the difference remaining less than 10% at large rotations. Due to the flexural deformations of the beam, this behavior would be expected [see Figure 6.101(c)].

Figure 6.100: Test 2 beam inclinometer rotations – (a) near beam south end, $\theta_{RT1}$; (b) near beam midspan, $\theta_{RT2}$. 
6.3.10 Load Block Displacements and Rotations

String pots DT3 – DT5 are used to measure the vertical $y$-displacements and the horizontal $x$-displacement of the load block. Similar to the vertical beam displacements, as the load block is displaced, the strings of the load block displacement transducers undergo a change in angle; and thus, their measurements may need to be adjusted to give the $x$- and $y$-displacements of the load block as described in Chapter 5.

Figures 6.102 through 6.104 show the measured displacements $\Delta_{DT3}$, $\Delta_{DT4}$, and $\Delta_{DT5}$, respectively, the corresponding adjusted displacements $\Delta_{DT3,x}$, $\Delta_{DT4,y}$, and $\Delta_{DT5,y}$,
respectively, and the percent differences between the measured and adjusted displacements. Note that, as described in Chapter 5, the negative $\Delta_{DT3}$ and $\Delta_{DT3,x}$ measurements indicate the movement of the load block in the north direction. It can be seen that the difference between $\Delta_{DT3}$ and $\Delta_{DT3,x}$ is well over 10% for much of the duration of the test; and thus, adjustments need to be applied to the measurements from DT3. In comparison, the adjustments needed for the vertical displacement measurements from DT4 and DT5 remain small throughout the test with the largest difference being less than 1.0%. Figure 6.105 shows the percent difference between the measured and adjusted displacements for DT4 and DT5 plotted against the load block beam chord rotation, $\theta_{lb}$. It can be seen that away from the origin, the maximum differences between the unadjusted and adjusted measurements from DT4 and DT5 remain less than 0.2%.

The measurements from DT3 require larger adjustments than those from DT4 and DT5 since the changes in the string angle for DT3, which occur due to the applied vertical displacements of the load block, are much larger than the changes in the string angles for DT4 and DT5, which occur due to the gap opening displacements at the beam ends. In evaluating the results from Test 2, adjusted measurements are used for $\Delta_{DT3}$; however, the measurements for $\Delta_{DT4}$ and $\Delta_{DT5}$ are not adjusted. Figure 6.105 plots $\Delta_{DT4}$, $\Delta_{DT5}$, and $\Delta_{DT3,x}$ against the load block beam chord rotation, $\theta_{lb}$. It can be seen in Figures 6.106(a) and 6.106(b) that the vertical displacements of the load block at the north and south ends are nearly the same.

Combining these displacements, the $x$-displacement, $y$-displacement, and rotation of the load block centroid can be determined as described in Chapter 5 and shown in Figures 6.107 and 6.108. Figure 6.106(a) plots the $y$-displacement versus the $x$-
displacement showing the path of the load block centroid during the test. As the subassembly is displaced under positive (i.e., clockwise) and negative (i.e., counterclockwise) rotations, the load block is pushed north (in the \(x\)-direction, away from the reaction block) due to the gap opening at the beam ends. After each cycle, the load block returns to its initial position with minimal residual displacements. Figures 6.107(a) and 6.108(a) show that the \(x\)-direction displacement of the load block centroid is slightly smaller during the negative rotations of the subassembly as compared to the positive rotations, which is possibly due to any unsymmetric loading or behavior of the structure. In the \(y\)-direction, the load block displaces symmetrically during the positive and negative rotations as shown in Figures 6.107(a), 6.107(d), and 6.108(b). Finally, the rotation of the load block is shown to remain small throughout the duration of the test as plotted in Figures 6.107(b) and 6.108(c). The load block rotation remains below 0.004 radians indicating that the two hydraulic actuators moved near simultaneously.
Figure 6.102: Test 2 load block horizontal displacements – (a) measured, $\Delta DT3$; (b) adjusted, $\Delta DT3,x$; (c) percent difference.
Figure 6.103: Test 2 load block north end vertical displacements – (a) measured, $\Delta DT_4$; (b) adjusted, $\Delta DT_{4,y}$; (c) percent difference.
Figure 6.104: Test 2 load block south end vertical displacements – (a) measured, $\Delta_{DT5}$; (b) adjusted, $\Delta_{DT5,y}$; (c) percent difference.
Figure 6.105: Test 2 percent difference between measured and adjusted displacements – (a) DT4; (b) DT5.

Figure 6.106: Test 2 load block displacements versus load block beam chord rotation – (a) $\Delta DT4 - \theta_{b,lb}$; (b) $\Delta DT5 - \theta_{b,lb}$; (c) $\Delta DT3,x - \theta_{b,lb}$. 
Figure 6.107: Test 2 load block centroid displacements – (a) x-y displacements; (b) rotation; (c) x-displacements; (d) y-displacements.
6.3.11 Reaction Block Displacements and Rotations

String pots DT6 – DT8 are used to measure the vertical y-displacements and the horizontal x-displacement of the reaction block. Figure 6.109 shows the measurements from DT6 – DT8 for the duration of the test. Since the reaction block is tied to the strong floor, the measured displacements remain very small throughout the test. Due to these small displacements and the use of lead cables for each string pot, the change in angle that the string undergoes during testing is very small. Thus, it can be assumed that no
adjustments are needed for the displacements measured from the reaction block displacement transducers. Figure 6.110 plots the measurements from DT6 – DT8 against the load block beam chord rotation.

The $x$-displacement, $y$-displacement, and rotation of the reaction block centroid can be determined (see Chapter 5) as shown in Figure 6.111 and plotted against the load block beam chord rotation in Figure 6.112. It is concluded that the vertical displacements of the reaction block do not have a significant effect on the displacements of the test structure (e.g., the beam chord rotation), and the test results are presented with the reaction block displacements taken as zero (i.e., the measured displacements of the reaction block are ignored in investigating the response of the subassembly). Note that the horizontal displacements of the reaction block are significant when determining the total elongation of the post-tensioning tendon; and thus, are included in those calculations.
Figure 6.109: Test 2 reaction block displacements – (a) $\Delta DT_6$; (b) $\Delta DT_7$; (c) $\Delta DT_8$. 
Figure 6.110: Test 2 reaction block displacements versus load block beam chord rotation – (a) $\Delta DT7 - \theta_{b,lb}$; (b) $\Delta DT8 - \theta_{b,lb}$; (c) $\Delta DT6 - \theta_{b,lb}$. 
Figure 6.111: Test 2 reaction block centroid displacements – (a) x-y displacements; (b) rotation; (c) x-displacements; (d) y-displacements.
Figure 6.112: Test 2 reaction block centroid displacements versus load block beam chord rotation – (a) x-displacements; (b) y-displacements; (c) rotation.

6.3.12 Contact Depth and Gap Opening at Beam-to-Wall Interfaces

The beam contact depth and gap opening displacements are measured at the beam-to-reaction-block interface using displacement transducers DT11 – DT13. As described in Chapter 5, these LVDTs rotate with the beam; and thus, their measurements may need to be adjusted to determine the gap opening displacements in the horizontal x-direction.
Figures 6.113 through 6.115 plot the measured displacements $\Delta_{\text{DT}11}$, $\Delta_{\text{DT}12}$, and $\Delta_{\text{DT}13}$, respectively, the corresponding adjusted $x$-displacements $\Delta_{\text{DT}11,x}$, $\Delta_{\text{DT}12,x}$, and $\Delta_{\text{DT}13,x}$, respectively, and the percent differences between the measured and adjusted displacements. The results indicate that the adjusted measurements are less than 0.3% different from the original measurements; and thus, $\Delta_{\text{DT}11}$, $\Delta_{\text{DT}12}$, and $\Delta_{\text{DT}13}$ can be taken as the displacements in the $x$-direction. Figure 6.116 plots the measured data from DT11, DT12, and DT13 against the load block beam chord rotation.

The maximum average concrete compressive strain in the beam-to-wall contact regions can be calculated by dividing the measured displacements from DT11 and DT13 with the gauge length (i.e., the distance from the LVDT ferrule insert in the beam to the reaction plate ferrule insert in the wall test region; see Chapter 5). For Test 2, the maximum average compressive strain is 0.0065. Note that this measurement includes the compressive strain occurring in the fiber-reinforced grout at the beam-to-wall interface. Also note that these results may have been affected by the loosening of the ferrule inserts as damage accumulated at the beam ends.
Figure 6.113: Test 2 beam-to-reaction-block interface top LVDT displacements – (a) measured, $\Delta_{DT11}$; (b) adjusted, $\Delta_{DT11,x}$; (c) percent difference.
Figure 6.114: Test 2 beam-to-reaction-block interface middle LVDT displacements – (a) measured, $\Delta_{DT12}$; (b) adjusted, $\Delta_{DT12,x}$; (c) percent difference.
Figure 6.115: Test 2 beam-to-reaction-block interface bottom LVDT displacements – (a) measured, $\Delta_{DT13}$; (b) adjusted, $\Delta_{DT13,x}$; (c) percent difference.
Using the measured data, the contact depth and the largest (i.e., at the beam top and bottom) gap opening displacements at the beam-to-reaction-block interface can be determined following the procedures in Chapter 5. Figures 6.117(a) and 6.118(a) show the results based on the measured data from DT11 – DT13 (method 1); Figures 6.117(b) and 6.118(b) show the results based on the measured data from RT2, DT11, and DT13 (method 2); Figures 6.117(c) and 6.118(c) show the results based on the measured data from RT2 and DT12 (method 3); Figures 6.117(d) and 6.118(d) show the results based on the load block beam chord rotation, $\theta_{b,lb}$ and the measurements from DT12 (method 4);
and Figures 6.117(e) and 6.118(e) show the results based on the load block beam chord rotation, $\theta_{b,lb}$ and the measurements from DT11 and DT13 (method 5). Note that in Chapter 5, methods 2 and 3 use the data from RT1; however, because this measurement is not available for Test 2, the data from RT2 is used. Each $\circ$ marker in Figures 6.117 and 6.118 indicates the contact depth or gap opening displacement at the peak of a loading cycle up to a beam chord rotation of 6.4%.

Looking at Figures 6.117 and 6.118, it can be stated that the contact depth and gap opening results obtained using the five methods are reasonably close, with the exception of the positive rotation contact depths calculated using methods 1, 2, and 5 due to the loosening of the ferrule insert for DT11. There is a rapid reduction in the contact depth up to a beam chord rotation of about 2.0%. After this rotation, the contact depth remains relatively stable due to the nonlinear behavior of the concrete in compression and varies between 4.0 – 2.0 in. (102 – 51 mm) [about 30 – 15% of the beam depth]. In comparison, the increase in gap opening with the rotation of the beam is very close to linear.

Figure 6.119 shows a continuous plot of the largest gap opening displacements determined from method 4 (using the load block beam chord rotation and DT12) against the load block beam chord rotation. Similarly, Figure 6.120(a) plots the beam contact depth from method 4 against the load block beam chord rotation as continuous data. Furthermore, Figures 6.120(b) and 6.120(c) show continuous plots of the contact depth during the 2.25% and 5.0% beam chord rotation cycles, respectively. It is observed that during the smaller displacement cycles of the subassembly (up through approximately 2.0% beam chord rotation), the beam is initially in full contact with the wall test region. As the system is loaded, a gap begins to open and a rapid reduction of the contact depth is
observed in Figure 6.120(b), and upon removal of the load, the beam-to-wall interface returns back to full contact. During the larger displacement cycles of the subassembly, there is an accumulation of damage at the beam ends including cracking of concrete and yielding of the angles in tension. This damage (e.g., cracks not fully closing, tension angles not fully yielding back in compression) distorts the LVDT measurements near the zero position [i.e., at small $\theta_{b,lb}$ values, see Figure 6.120(c)]. As the beam is rotated, the cracks in the compression zone close further and the contact depth measurement approaches the behavior observed during the small rotation cycles. Note that the gap opening and contact depth plots in Figures 6.118 and 6.119 could have been affected by the use of $\theta_{b,lb}$ (i.e, chord rotation) instead of $\theta_{RT1}$ (i.e., local rotation) to determine the behavior at the south end of the beam. The use of the beam midheight transducer DT12 instead of the extreme beam top or bottom transducer (DT11 or DT13) may also have affected the results, especially the contact depth plots in Figure 6.119 during small beam rotations, which are very sensitive to the measurements. Thus, the estimated contact depths at small beam rotations ($\theta_{b} < 0.25\%$) should be used with caution.

In addition to the LVDT measurements, the gap opening measurements at the south end of the beam were taken using a ruler. These ruler measurements are listed in Table 6.2 and shown using $\pm$ markers in Figure 6.118. It can be seen that the LVDT gap opening measurements (○ markers) in Figure 6.118 are reasonably close to the ruler measurements. Note that no ruler measurements were taken after a beam chord rotation of 3.33% due to the beam end damage.
Figure 6.117: Test 2 contact depth at beam-to-reaction-block interface –
(a) method 1 using DT11, DT12, and DT13; (b) method 2 using RT2, DT11, and DT13;
(c) method 3 using RT2 and DT12; (d) method 4 using $\theta_{b,lb}$ and DT12; (e) method 5
using $\theta_{b,lb}$, DT11, and DT13.
Figure 6.118: Test 2 gap opening at beam-to-reaction-block interface –  
(a) method 1 using DT11, DT12, and DT13; (b) method 2 using RT2, DT11, and DT13;  
(c) method 3 using RT2 and DT12; (d) method 4 using $\theta_{b,lb}$ and DT12; (e) method 5  
using $\theta_{b,lb}$, DT11, and DT13.
Figure 6.119: Test 2 gap opening at beam-to-reaction-block interface using method 4.

Figure 6.120: Test 2 contact depth at beam-to-reaction-block interface using method 4 – (a) entire data; (b) 2.25% beam chord rotation cycle; (c) 5.0% beam chord rotation cycle.
TABLE 6.2

RULER MEASUREMENTS OF GAP OPENING AT SOUTH BEAM END

<table>
<thead>
<tr>
<th>Nominal Rotation (%)</th>
<th>Gap Opening, ( \Delta g ) [in. (mm)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>visible</td>
</tr>
<tr>
<td>0.35</td>
<td>0.016 (0.41)</td>
</tr>
<tr>
<td>0.50</td>
<td>0.02 (0.51)</td>
</tr>
<tr>
<td>0.75</td>
<td>0.06 (1.52)</td>
</tr>
<tr>
<td>1.0</td>
<td>0.09375 (2.38)</td>
</tr>
<tr>
<td>1.5</td>
<td>0.125 (3.175)</td>
</tr>
<tr>
<td>2.25</td>
<td>0.15625 (3.97)</td>
</tr>
<tr>
<td>3.33</td>
<td>0.375 (9.53)</td>
</tr>
<tr>
<td>5.0</td>
<td>no measurements taken due to beam end damage</td>
</tr>
<tr>
<td>6.4</td>
<td></td>
</tr>
</tbody>
</table>

6.3.13 Wall Test Region Local Concrete Deformations

The reaction block confined concrete deformations near the beam-to-wall interface of the wall test region are measured using displacement transducers DT14 and DT15. Figure 6.121 plots the time history results from the top (DT14) and bottom (DT15) transducers, and Figure 6.122 plots the measured data from DT14 and DT15 against the load block beam chord rotation. As expected, the concrete deformations are mostly compressive (negative) due to the compression stresses that are transferred through the contact region from post-tensioning.

From Figure 6.121, the maximum average concrete compressive strain in the wall test region can be calculated by dividing the measured displacements with the gauge.
length (i.e., the distance from the LVDT ferrule insert in the wall test region to the reaction plate; see Chapter 5). For Test 2, the maximum average concrete compressive strain is 0.0037, which is close to but below the expected unconfined (cover) concrete crushing strain of 0.004. This finding is in accordance with the visual observation that only minor spalling of the cover concrete occurred in the wall test region during the test.

Figure 6.121: Test 2 wall test region concrete deformations – (a) DT14; (b) DT15.
6.3.14 Beam Looping Reinforcement Longitudinal Leg Strains

Figures 6.123 and 6.124 show the strain gauge measurements for the top and bottom horizontal legs of the east and west No. 6 mild steel looping reinforcing bars in the beam. The locations of these strain gauges can be found in Chapter 5. The initial beam post-tensioning force results in a small compressive strain in the bars at the beginning of the test. Upon lateral loading of the subassembly, the largest tensile strains occur, as expected, in the gauges closest to the angle-to-beam connection bolts [i.e., gauges 6(1)T-E, 6(1)T-W, 6(1)B-E, and 6(1)B-W]. The measurements in the gauges away from the angle-to-beam connection decrease with distance from this critical location.

To provide a better understanding of the strain measurements in the horizontal legs of the beam looping reinforcement, the $\Delta$ and $\Box$ markers in Figures 6.123(a) and 6.123(b) for gauges 6(1)T-E and 6(1)T-W correspond to positive and negative chord rotation peaks for the beam, respectively, and the $\circ$ markers indicate zero rotation
positions. To provide further insight into the results, Figures 6.125 and 6.126 show the strains plotted against the beam chord rotation. In the positive (i.e., clockwise) rotation direction, the strains in the top bars increase in tension as the gap opens at the top south corner of the beam and the top angle is pulled in tension. In the negative (i.e., counterclockwise) direction, the strains in the bottom bars increase in tension and the top bars go into compression due to the closing of the gap.

The maximum strains in gauges 6(1)T-W and 6(1)B-E are close to and below the yield strain of the longitudinal steel ($\varepsilon_{ty} = 0.00283$) from the material tests in Chapter 4; and thus, it is concluded that the amount of mild steel reinforcement used to transfer the angle forces into the beam is adequate.
Figure 6.123: Test 2 beam looping reinforcement top longitudinal leg strains –
(a) $\varepsilon_{6(1)T-E}$; (b) $\varepsilon_{6(1)T-W}$; (c) $\varepsilon_{6(2)T-E}$; (d) $\varepsilon_{6(2)T-W}$; (e) $\varepsilon_{6(3)T-E}$; (f) $\varepsilon_{6(3)T-W}$; (g) $\varepsilon_{6MT-E}$; (h) $\varepsilon_{6MT-W}$. 
Figure 6.123 continued.

Figure 6.124: Test 2 beam looping reinforcement bottom longitudinal leg strains –
(a) $\varepsilon_{6(1)B-E}$; (b) $\varepsilon_{6(1)B-W}$; (c) $\varepsilon_{6(2)B-E}$; (d) $\varepsilon_{6(2)B-W}$; (e) $\varepsilon_{6(3)B-E}$; (f) $\varepsilon_{6(3)B-W}$; (g) $\varepsilon_{6MB-E}$; (h) $\varepsilon_{6MB-W}$. 
Figure 6.124 continued.

Figure 6.125: Test 2 beam looping reinforcement top longitudinal leg strains versus load block beam chord rotation – (a) $\varepsilon_{6(1)T-E}$-$\theta_{b,lb}$; (b) $\varepsilon_{6(1)T-W}$-$\theta_{b,lb}$; (c) $\varepsilon_{6(2)T-E}$-$\theta_{b,lb}$; (d) $\varepsilon_{6(2)T-W}$-$\theta_{b,lb}$; (e) $\varepsilon_{6(3)T-E}$-$\theta_{b,lb}$; (f) $\varepsilon_{6(3)T-W}$-$\theta_{b,lb}$; (g) $\varepsilon_{6MT-E}$-$\theta_{b,lb}$; (h) $\varepsilon_{6MT-W}$-$\theta_{b,lb}$.
Figure 6.125 continued.
Figure 6.126: Test 2 beam looping reinforcement bottom longitudinal leg strains versus load block beam chord rotation – (a) $\varepsilon_{6(1)B-E}$-$\theta_{b,lb}$; (b) $\varepsilon_{6(1)B-W}$-$\theta_{b,lb}$; (c) $\varepsilon_{6(2)B-E}$-$\theta_{b,lb}$; (d) $\varepsilon_{6(2)B-W}$-$\theta_{b,lb}$; (e) $\varepsilon_{6(3)B-E}$-$\theta_{b,lb}$; (f) $\varepsilon_{6(3)B-W}$-$\theta_{b,lb}$; (g) $\varepsilon_{6MB-E}$-$\theta_{b,lb}$; (h) $\varepsilon_{6MB-W}$-$\theta_{b,lb}$.
6.3.15 Beam Transverse Reinforcement Strains

Figure 6.127 shows the strain measurements from gauges 6SE(I)-E, 6SE(E)-E, 6SE(I)-W, and 6SE(E)-W placed on the transverse (i.e., vertical) legs of the east and west No. 6 looping reinforcing bars at the south end of the beam. The locations of these strain gauges can be found in Chapter 5. To give more insight into the measurements, Figure 6.128 plots the strain data against the load block beam chord rotation. It can be seen that the strain gauge readings remain well below the yield strain of the reinforcing steel \( \varepsilon_{\text{y}} = 0.00283 \) throughout the experiment, demonstrating that the design of the transverse reinforcement at the beam ends is adequate. Note that the angle-to-beam connection bolts may also have acted as transverse reinforcement in the beam; however, this could not be confirmed from the test results since the connection bolts were not instrumented.

Similarly, Figure 6.129 shows the strain measurements from gauges MH-E and MH-W placed on the vertical legs of the No. 3 transverse hoop at the beam midspan. To
give more insight into the measurements, Figure 6.130 plots the strain data against the load block beam chord rotation. As expected, the results indicate that the maximum strains in the midspan hoop remained well below the yield strain ($\varepsilon_{dy} = 0.00240$, see Chapter 4); and thus, the use of nominal transverse reinforcement within the span of the beam is adequate.

![Figure 6.127: Test 2 beam looping reinforcement vertical leg strains – (a) $\varepsilon_{6SE(I)-E}$; (b) $\varepsilon_{6SE(I)-W}$; (c) $\varepsilon_{6SE(I)-E}$; (d) $\varepsilon_{6SE(I)-W}$.](image)
Figure 6.128: Test 2 beam looping reinforcement vertical leg strains versus load block beam chord rotation – (a) $\varepsilon_{6SE(E)-E} - \theta_{b,lb}$; (b) $\varepsilon_{6SE(E)-W} - \theta_{b,lb}$; (c) $\varepsilon_{6SE(I)-E} - \theta_{b,lb}$; (d) $\varepsilon_{6SE(I)-W} - \theta_{b,lb}$.

Figure 6.129: Test 2 beam midspan transverse hoop reinforcement strains – (a) $\varepsilon_{MH-E}$; (b) $\varepsilon_{MH-W}$. 

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Figure 6.130: Test 2 beam midspan transverse hoop reinforcement strains versus load block beam chord rotation – (a) $\epsilon_{MH-E}-\theta_{b,lb}$; (b) $\epsilon_{MH-W}-\theta_{b,lb}$.

6.3.16 Beam Confined Concrete Strains

Figures 6.131 and 6.132 show the measurements from the strain gauges placed on the No. 3 support bars inside the hoop confined concrete at the south end of the beam. The locations of these strain gauges can be found in Chapter 5. For further insight into the strain gauge readings, Figures 6.133 and 6.134 plot the strain data against the load block beam chord rotation. The results show that there is a small compressive strain at the beginning of the test due to the initial post-tensioning force. As the beam is rotated in the positive (i.e., clockwise) direction, the compression strains in the bottom bars increase due to the transfer of the contact stresses through the bottom corner of the beam. In the opposite (i.e., counterclockwise) direction, the strains in the top and bottom bars reverse due to the reversal of the load.
Since the strain gauge measurements remain below the yield strain $\varepsilon_{hy} = 0.00240$ of the No. 3 support bars in tension and below the expected crushing strain $\varepsilon_{cu} = 0.004$ of the unconfined concrete in compression, it can be stated that these measurements are not representative of the amount of damage observed at the beam ends, possibly due to the deterioration of bond between the steel and the concrete.

Figure 6.131: Test 2 No. 3 top hoop support bar strains—
(a) $\varepsilon_{3THT-(1)}$; (b) $\varepsilon_{3THB-(1)}$; (c) $\varepsilon_{3THT-(2)}$; (d) $\varepsilon_{3THB-(2)}$. 
Figure 6.132: Test 2 No. 3 bottom hoop support bar strains – (a) $\varepsilon_{3BHB-(1)}$; (b) $\varepsilon_{3BHT-(1)}$; (c) $\varepsilon_{3BHB-(2)}$; (d) $\varepsilon_{3BHT-(2)}$. 

strain gauge 3BHT-(1) removed from Beam 2

strain gauge 3BHT-(2) removed from Beam 2
Figure 6.133: Test 2 No. 3 top hoop support bar strains versus load block beam chord rotation – (a) $\varepsilon_{3THT-(1)} - \theta_{b,lb}$; (b) $\varepsilon_{3THB-(1)} - \theta_{b,lb}$; (c) $\varepsilon_{3THT-(2)} - \theta_{b,lb}$; (d) $\varepsilon_{3THB-(2)} - \theta_{b,lb}$. 
6.3.17 Beam End Confinement Hoop Strains

Figures 6.135 and 6.136 show the measurements from the strain gauges placed on the vertical legs of the bottom layer No. 3 confinement hoops at the south end of the beam. The locations of these strain gauges can be found in Chapter 5. For further insight into the measurements, Figures 6.137 and 6.138 plot the strain data against the load block beam chord rotation. The angle-to-beam connection bolt forces result in a small compressive strain in the vertical hoop steel at the beginning of the test. Throughout
much of the test, the measured confinement hoop strains remain small. Strain gauges 4HB-E, 1HB-W, and 2HB-W experience a large increase in compressive strain near the end of the test, which is most likely due to the damage at the end of the beam. Similar to the behavior observed in Test 1 (see Figure 6.67), the significant loss of cover concrete around the hoops and the presence of high compressive stresses in the vertical direction caused the vertical legs of the hoops to bend outward, resulting in the strain gauge measurements to be distorted. Note that the strain gauges were placed on the inside of the vertical legs, leading to the development of compression strains due to the outward bending of the hoop reinforcement.

Figure 6.135: Test 2 beam end confinement hoop east leg strains –
(a) $\varepsilon_{1HB-E}$; (b) $\varepsilon_{2HB-E}$; (c) $\varepsilon_{3HB-E}$; (d) $\varepsilon_{4HB-E}$. 
Figure 6.136: Test 2 beam end confinement hoop west leg strains –
(a) $\varepsilon_{1HB-W}$; (b) $\varepsilon_{2HB-W}$; (c) $\varepsilon_{3HB-W}$; (d) $\varepsilon_{4HB-W}$. 
Figure 6.137: Test 2 beam end confinement hoop east leg strains versus load block beam chord rotation – (a) $\varepsilon_{1HBE}$-$\theta_{b,lb}$; (b) $\varepsilon_{2HBE}$-$\theta_{b,lb}$; (c) $\varepsilon_{3HBE}$-$\theta_{b,lb}$; (d) $\varepsilon_{4HBE}$-$\theta_{b,lb}$. 
6.3.18 Wall Test Region Confined Concrete Strains

As described previously, the strain gauge wires coming out of the reaction block used in Tests 2 – 4B were all severed during the removal of the steel casting mold. Thus, no measurements were recorded for the wall test wall region confined concrete strains in Test 2.
6.3.19 Wall Test Region Confinement Hoop Strains

Similar to above, no measurements were recorded for the wall test region confinement hoop strains in Test 2 since the strain gauge wires were severed.

6.3.20 Crack Patterns

The crack propagation from testing is marked on the beam during each cycle of displacement (e.g., see Figures 6.77, 6.78, and 6.79). In addition, the crack propagation is recorded by approximately copying the crack patterns manually onto paper. Figure 6.139 shows the hand-drawn crack patterns recorded for Test 2 at the end of the last cycle from selected displacement increments (see Appendix F for crack sheets).

Figure 6.139: Test 2 crack patterns – (a) $\theta_b = 0.5\%$; (b) $\theta_b = 0.75\%$; (c) $\theta_b = 1.0\%$; (d) $\theta_b = 1.5\%$; (e) $\theta_b = 2.25\%$; (f) $\theta_b = 3.33\%$; (g) $\theta_b = 5.0\%$; (h) $\theta_b = 6.4\%$. 
Figure 6.139 continued.
6.4 Test 3

The beam used in Test 3 (i.e., Beam 3) has the following properties: (1) beam depth, \( h_b = 14 \text{ in.} \) (356 mm); (2) beam width, \( b_b = 7.5 \text{ in.} \) (191 mm); (3) mild steel reinforcement of two No. 6 bars looping around the beam perimeter along its length; (4) No. 3 full-depth rectangular hoops [6.125 in. by 12.675 in. (156 mm by 322 mm)] placed at a nominal 7.0 in. (178 mm) spacing to provide transverse reinforcement in the beam midspan region; (5) No. 3 partial-depth rectangular hoops [6.125 in. by 4.375 in. (156 mm by 111 mm)] placed at a 1.5 in. (38 mm) spacing to provide concrete confinement at the beam ends; (6) a beam post-tensioning tendon comprised of three 0.6 in. (15 mm) nominal diameter high-strength strands with a total area of \( A_{bp} = 0.651 \text{ in.}^2 \) (420 mm\(^2\)); (7) average initial beam post-tensioning strand stress of \( f_{bpi} = 0.39 f_{bpu} \), where \( f_{bpu} = 270 \text{ ksi} \) (1862 MPa) is the design maximum strength of the post-tensioning steel; (8) total initial beam post-tensioning force of \( P_{bi} = 69.4 \text{ kips} \) (309 kN); (9) initial beam concrete nominal axial stress (based on actual cross-sectional area with beam post-tensioning duct removed) of \( f_{bc} = 0.68 \text{ ksi} \) (4.7 MPa); and (10) two top and two seat angles (L8x8x1/2) comprised of 2 – 2.5 in. (64 mm) long angle strips resulting in a total angle length of \( l_a = 5.0 \text{ in.} \) (127 mm).

The primary parameter differences of Test 3 from Test 2 are: (1) reduced beam post-tensioning tendon area; (2) reduced initial beam concrete nominal axial stress; and (3) reduced angle strength. The same displacement loading history from Test 2 was used for Test 3. Furthermore, the measures taken to prevent premature wire fractures in the
beam post-tensioning strands in Test 2 were also used in Test 3, as well as the use of a reduced height grout column at the beam-to-wall interfaces.

6.4.1 Test Photographs

Photographs of the original and displaced subassembly configurations from Test 3 are shown in Figures 6.140 and 6.141. Figures 6.140(a) through 6.140(f) show overall subassembly photographs as follows: (a) pre-test undisplaced position; (b) displaced to $\theta_b = 3.33\%$; (c) displaced to $\theta_b = -3.33\%$; (d) displaced to $\theta_b = 5.0\%$; (e) displaced to $\theta_b = -5.0\%$; and (f) final post-test undisplaced position. Similarly, Figures 6.141(a) through 6.141(f) show close-up photographs of the south end of the beam at the beam-to-reaction-block interface. The accumulation of damage at the south end of the beam is shown in more detail in Figure 6.142.

The use of two short angle strips (see Figure 6.143) led to damage in the wall test region of the reaction block. A single post-tensioning strand was used on each angle strip to connect the angles to the walls. This applied a force similar to a concentrated point load on the wall test region due to the reduction of the area over which the force was applied. Spalling of the cover concrete at the beam ends initiated at a beam chord rotation of $\theta_b = 1.5\%$; however, the spalling was relatively small up to a beam chord rotation of $\theta_b = 5.0\%$. As discussed later, the damage in the wall test region of the reaction block played a significant role on the behavior of the system. The test was stopped after 5.0% beam chord rotation to prevent further damage to the wall test region.

The angle-to-wall connection strands performed well with no yielding; however, the reduced angle length led to concrete damage in the wall test region as described
above. Slip between the coupling beam and the walls did not occur demonstrating that the friction resistance due to the post-tensioning force provided adequate vertical support to the beam together with resistance from the top and seat angles. Similar to Test 2, premature wire fracture in the post-tensioning strands did not occur in Test 3. Furthermore, no slip in the angle-to-beam connections was observed during the test (unlike Tests 1 and 2), possibly due to the reduced amount of damage to the concrete at the beam ends.

Under smaller displacements of the coupling beam up to about 2.25% rotation, gap opening occurred between the fiber-reinforced grout and the faces of the load and reaction blocks. Upon further displacements of the structure, gap opening continued to form until the end of the test; however, it was observed that gaps formed on both sides of the grout column. While this is not the desired mode of behavior for the structure as described in Chapter 3, it did not change the behavior of the system or cause any problems with the performance of the grout. No significant crushing of the grout at the beam-to-wall interfaces was observed throughout the test. Note that the grout column was left approximately 0.25 in. (6.4 mm) short of the beam depth at the top and bottom, as was also done in Test 2.
Figure 6.140: Test 3 overall photographs – (a) pre-test undisplaced position; 
(b) $\theta_b = 3.33\%$; (c) $\theta_b = 3.33\%$; (d) $\theta_b = 5.0\%$; (e) $\theta_b = -5.0\%$; 
(f) final post-test undisplaced position.
Figure 6.141: Test 3 beam south end photographs – (a) pre-test undisplaced position; (b) $\theta_b = 3.33\%$; (c) $\theta_b = -3.33\%$; (d) $\theta_b = 5.0\%$; (e) $\theta_b = -5.0\%$; (f) final post-test undisplaced position.
Figure 6.142: Test 3 beam south end damage propagation – positive and negative rotations.

Figure 6.143: Test 3 angle strips at south top connection.
6.4.2 Beam Shear Force versus Chord Rotation Behavior

Figure 6.144(a) shows the hysteretic coupling beam shear force, $V_b$ versus chord rotation, $\theta_b$ behavior from Test 3, where $V_b$ and $\theta_b$ are calculated from Equations 5.5 and 5.35, respectively. For comparison, Figure 6.144(b) plots the $V_b$-$\theta_{b,ib}$ behavior, where $\theta_{b,ib}$ is the beam chord rotation determined from the vertical ($y$-direction) displacement of the load block centroid using Equation 5.36. As shown in Figure 6.144, the structure was able to sustain three cycles at 5.0% rotation with approximately 12.5% loss in shear resistance. This loss was due to the damage occurring to the concrete in the wall test region of the reaction block, rather than the damage in the coupling beam. The test was stopped at this point (prior to the failure of the beam) to prevent any further damage so that the reaction block could be used in the following experiments.

Looking at the hysteresis loops, it can be seen that the specimen was able to dissipate a considerable amount of energy. Most of this energy dissipation occurred due to the yielding of the top and seat angles. The post-tensioning tendon did not have any wire fracture during the test due to the reduced initial stress in the post-tensioning strands and the use of additional anchor barrels to reduce strand “kinking” inside the anchors (see Section 6.4.4). The beam had a sufficient amount of restoring force to yield the tension angles back in compression and close the gaps at the beam-to-wall interfaces upon unloading, resulting in a large self-centering capability.
Figure 6.144: Test 3 coupling beam shear force versus chord rotation behavior – (a) using beam displacements; (b) using load block displacements.
6.4.3 Beam End Moment Force versus Chord Rotation Behavior

Figure 6.145 shows the hysteretic coupling beam end moment, $M_b$, versus chord rotation, $\theta_b$ behavior from Test 3, where $M_b$ is determined from Equation 5.6 as described in Chapter 5. Since the beam end moment is calculated from the beam shear force, the results shown in Figure 6.145 are directly related to the results in Figure 6.144; and thus, no further discussion is provided herein.

![Figure 6.145: Test 3 $M_b-\theta_b$ behavior.](image)

6.4.4 Beam Post-Tensioning Forces

The coupling beam post-tensioning strand forces from the test are measured using load cells LC15 – LC17 mounted at the dead ends of the three strands (see Chapter 5).
The forces from the three load cells, $F_{LC15}$, $F_{LC16}$, and $F_{LC17}$ are shown in Figure 6.144, and are plotted against the beam chord rotation, $\theta_b$ in Figure 6.145. Figure 6.146 shows the total beam post-tensioning tendon force, $P_{bp}$ (sum of the forces in the three strands) normalized with the total design ultimate strength of the tendon, $\Sigma a_{bp} f_{bpu}$. The total initial beam post-tensioning tendon force is equal to 69.4 kips (309 kN), resulting in an initial beam concrete nominal axial stress of $f_{bci} = 0.68$ ksi (4.7 MPa).

As the structure is displaced and gap opening occurs at the beam ends, the post-tensioning strands elongate and the post-tensioning forces increase. Since the strands are unbonded over the entire length of the subassembly, the nonlinear straining of the post-tensioning steel is significantly delayed, with only small losses occurring in the post-tensioning forces up through the 3.33% beam chord rotation cycles (possibly due to small amounts of nonlinear straining of the post-tensioning steel, additional seating of the anchor wedges, and/or nonlinear behavior in the beam/wall concrete). During the 5.0% rotation cycles, additional losses are observed in the post-tensioning forces due to the concrete damage in the wall test region of the reaction block.

Note that similar steps were taken in Test 3 as in Test 2 to help prevent premature wire fractures in the post-tensioning strands, including the use of reduced initial post-tensioning stresses and the use of extra anchor barrels to reduce strand “kinking” inside the anchor wedges (see Section 6.3.4). These measures were successful to achieve satisfactory performance of the strand/anchor system during the experiment.
Figure 6.146: Test 3 beam post-tensioning strand forces – (a) strand 1; (b) strand 2; (c) strand 3.
Figure 6.147: Test 3 $F_{LC}$-$\theta_b$ behavior — (a) strand 1; (b) strand 2; strand 3.
Figure 6.148: Test 3 beam post-tensioning force versus chord rotation behavior – (a) using beam displacements; (b) using load block displacements.
6.4.5 Angle-to-Wall Connection Post-Tensioning Forces

The beam south end (i.e., reaction block end) angle-to-wall connection post-tensioning strand forces, \( F_{LC3} - F_{LC6} \), measured using load cells LC3 – LC6, respectively, are shown in Figures 6.147 through 6.150. The target initial force for each connection strand is 20 kips (89 kN); whereas, the measured initial forces in the four strands are \( F_{i,LC3} = 32.3 \text{ kips (144 kN)}, \) \( F_{i,LC4} = 17.2 \text{ kips (76 kN)}, \) \( F_{i,LC5} = 24.2 \text{ kips (108 kN)}, \) and \( F_{i,LC6} = 29.7 \text{ kips (132 kN)}. \) A significant variation is observed in the initial connection strand forces since even a slight difference in the amount of anchor wedge seating has a large effect on the initial force (due to the short length of the strands).

Figures 6.151(a) and 6.151(b) show the total forces in the south top and south seat angle connection strands, respectively, plotted against the beam chord rotation, \( \theta_b \). The connection forces are normalized with the total design ultimate strength of the strands, \( P_{abu} = \sum a_{ap} f_{apu} \), where \( f_{apu} = 270 \text{ ksi (1862 MPa)} \). The expected behavior of the strands is that as the structure is displaced and the angles are pulled in tension, the connection forces increase; and upon unloading, the connection forces return more or less back to the initial forces with possibly some losses occurring due to additional seating of the anchor wedges and any permanent deformations in the concrete (note that the nonlinear straining of the post-tensioning steel is prevented since the strands are left unbonded). These trends are seen in the top angle load cell LC4; however, the forces from the other load cells do not behave as expected. This could have been due to the malfunctioning of the load cells under the non-uniform loads applied during the prying deformations of the angles. Note that there is a drop in the load cell forces during the larger displacement cycles, with LC6
experiencing the largest drop in force, due to the damage occurring in the wall test region.

Figure 6.149: Test 3 south end top angle-to-wall connection strand forces – (a) east strand; (b) west strand.
Figure 6.150: Test 3 south end top angle-to-wall connection strand forces versus beam chord rotation – (a) $F_{LC3}\theta_b$; (b) $F_{LC4}\theta_b$.

Figure 6.151: Test 3 south end seat angle-to-wall connection strand forces – (a) east strand; (b) west strand.
Figure 6.152: Test 3 south end seat angle-to-wall connection strand forces versus beam chord rotation – (a) $F_{LC5}$-$\theta_b$; (b) $F_{LC6}$-$\theta_b$.

Figure 6.153: Test 3 south end total angle-to-wall connection strand forces versus beam chord rotation– (a) top connection; (b) seat connection.
6.4.6 Vertical Forces on Wall Test Region

Load cells LC7 – LC14 are used to measure the forces in the eight vertical bars applying axial compression forces to the wall test region of the reaction block and anchoring the block to the strong floor. The total vertical force, $F_{wt}$, is determined as described in Chapter 5, with the target initial total force ranging between 150 – 160 kips (667 – 712 kN). Figure 6.152(a) shows $F_{wt}$ for the duration of the test and Figure 6.152(b) plots $F_{wt}$ against the beam chord rotation, $\theta_b$. The initial total force, $F_{wt,i}$, is 132 kips (588 kN), below the target force range. As the beam is rotated in the positive (i.e., clockwise) direction with the load block moving down, $F_{wt}$ decreases since the beam applies a downward force on the reaction block. Similarly, as the beam is rotated in the negative (i.e., counterclockwise) direction, $F_{wt}$ increases since the beam applies an upward force on the reaction block. Note that, as described in Chapter 5, the amount of variation in $F_{wt}$ during the cyclic displacements of the beam is relatively small as compared with the expected variation of axial forces in the wall pier coupling regions of a multi-story coupled wall system. Upon unloading, $F_{wt}$ returns more or less to its initial value.

![Figure 6.154: Test 3 vertical force on wall test region, $F_{wt}$ - (a) $F_{wt}$-test duration; (b) $F_{wt}$-$\theta_b$](image-url)
6.4.7 Beam Vertical Displacements

The vertical displacements $\Delta_{DT9}$ and $\Delta_{DT10}$ at the south and north ends of the beam are measured using string pots DT9 and DT10, respectively. These displacements are used to calculate the beam chord rotation, $\theta_b$. As described in Chapter 5, as the subassembly is displaced, the transducer string undergoes a change of angle, which can be “adjusted” to give the vertical displacements in the $y$-direction. Figures 6.153 and 6.154 show the measured displacements $\Delta_{DT9}$ and $\Delta_{DT10}$, respectively, the corresponding adjusted $y$-displacements $\Delta_{DT9,y}$ and $\Delta_{DT10,y}$, respectively, and the difference between the measured and adjusted displacements for the duration of the test. Note that there is a negligible amount of drift [approximately 0.02 in. (0.51 mm)] in the data from DT9 indicating the beam might have shifted in the vertical direction with respect to the wall; however, it is not clear if this was due to slip and/or repositioning of the beam since no visual observations were made.

It can be seen from Figures 6.153 and 6.154 that $\Delta_{DT9}$ and $\Delta_{DT10}$ are close to $\Delta_{DT9,y}$ and $\Delta_{DT10,y}$, respectively, with the difference being less than 0.005 in. (0.13 mm). Figure 6.155 plots the percent difference between the measured and adjusted displacements versus the beam chord rotation, $\theta_b$. The results indicate that the largest percent differences occur when the beam chord rotation is close to zero; however, these differences are not significant since the corresponding measurements are very small and are mostly outside the sensitivity of the transducers. As the structure is displaced, the measurements from DT9 require larger adjustments than those from DT10, since corresponding to a given $\theta_b$, $\Delta_{DT9}$ is smaller than $\Delta_{DT10}$. It is also observed that for negative rotations, DT9 displays larger percent errors than under positive rotations because the measured displacements
under negative rotations are smaller (possibly due to the drift in the data) than the measured displacements under positive rotations. Therefore, similar differences between the measured and adjusted displacements under the negative and positive directions result in larger percent errors in the negative direction. Since the adjustments described in Chapter 5 require certain assumptions and approximations and since the amplitude differences (which are more important than percent differences for the calculations of the beam chord rotation) between the measured and adjusted displacements remain small, these differences are ignored and the measurements from DT9 and DT10 are used as the vertical $y$-displacements of the beam throughout this dissertation. Figure 6.156 plots the measured data from DT9 and DT10 against the beam chord rotation, $\theta_b$.

![Figure 6.156: Test 3 south end beam vertical displacements – (a) measured, $\Delta DT9$; (b) adjusted, $\Delta DT9,y$; (c) difference, $\Delta DT9,y - \Delta DT9$.](image)

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Figure 6.156: Test 3 north end beam vertical displacements – (a) measured, $\Delta_{DT10}$; (b) adjusted, $\Delta_{DT10,y}$; (c) difference, $\Delta_{DT10,y} - \Delta_{DT10}$.

Figure 6.157: Test 3 percent difference between measured and adjusted displacements versus beam chord rotation – (a) south end, DT9; (b) north end, DT10.
6.4.8 Beam Chord Rotation

The beam chord rotation is defined as the relative vertical displacement of the beam ends divided by the beam length. The beam chord rotation $\theta_b$ determined based on the $\Delta DT9$ and $\Delta DT10$ measurements in Test 3 is shown in Figure 6.157(a). For comparison, Figure 6.157(b) shows the load block beam chord rotation, $\theta_{b,lb}$ calculated using $\Delta LB,y$, and Figure 6.158 shows the percent difference between $\theta_b$ and $\theta_{b,lb}$ plotted against $\theta_b$. It can be seen that there is a large percent difference for small beam chord rotations, with the difference dropping down to less than 10% at larger beam rotations. Figure 6.159 plots the beam chord rotation, $\theta_b$ and the load block beam chord rotation, $\theta_{b,lb}$ against the beam chord rotation, $\theta_b$. The results show that the two rotations are nearly identical for both positive and negative rotations.
Figure 6.159: Test 3 beam chord rotation — (a) $\theta_b$ from $\Delta_{DT9}$ and $\Delta_{DT10}$; (b) $\theta_{b,lb}$ from $\Delta_{LB,y}$.

Figure 6.160: Test 3 percent difference between $\theta_b$ and $\theta_{b,lb}$.
Local Beam Rotations

Local beam rotations were measured using two rotation transducers (inclinometers); one near the midspan of the beam (RT1) and the other near the south end (RT2). Figure 6.160 shows the rotation time history results, $\theta_{RT1}$ and $\theta_{RT2}$ from these transducers. The rotation measurements are positive when the load block is displaced in the downward direction (i.e., clockwise beam rotation). When compared to the beam chord rotations calculated from the beam end displacements as shown in Figure 6.161(a), the beam south end rotations from RT1 are larger for negative rotations, but smaller for positive rotations. For the beam midspan, Figure 6.161(b) shows that the rotations from RT2 are larger than the beam chord rotations, with the difference remaining less than 10% at large rotations. Note that due to the flexural deformations of the beam as shown in Figure 6.161(c), it would be expected that the beam chord rotation, $\theta_b$ is smaller than
$\theta_{RT2}$ but larger than $\theta_{RT1}$. This behavior is not observed in Figure 6.161(a) for $\theta_{RT1}$ in the negative direction.

Figure 6.162: Test 3 beam inclinometer rotations – (a) near beam south end, $\theta_{RT1}$; (b) near beam midspan, $\theta_{RT2}$.
6.4.10 Load Block Displacements and Rotations

String pots DT3 – DT5 are used to measure the vertical y-displacements and the horizontal x-displacement of the load block. Similar to the vertical beam displacements, as the load block is displaced, the strings of the load block displacement transducers undergo a change in angle; and thus, their measurements may need to be adjusted to give the x- and y-displacements of the load block as described in Chapter 5.

Figures 6.162 through 6.164 show the measured displacements $\Delta_{DT3}$, $\Delta_{DT4}$, and $\Delta_{DT5}$, respectively, the corresponding adjusted displacements $\Delta_{DT3,x}$, $\Delta_{DT4,y}$, and $\Delta_{DT5,y}$, respectively, and the percent differences between the measured and adjusted
displacements. Note that, as described in Chapter 5, the negative $\Delta_{DT3}$ and $\Delta_{DT3,x}$ measurements indicate the movement of the load block in the north direction. It can be seen that the difference between $\Delta_{DT3}$ and $\Delta_{DT3,x}$ is well over $10\%$ for much of the duration of the test; and thus, adjustments need to be applied to the measurements from DT3. In comparison, the adjustments needed for the vertical displacement measurements from DT4 and DT5 remain small throughout the test with the largest difference being less than $0.4\%$. Figure 6.165 shows the percent difference between the measured and adjusted displacements for DT4 and DT5 plotted against the beam chord rotation, $\theta_b$. It can be seen that away from the origin, the maximum differences between the unadjusted and adjusted measurements from DT4 and DT5 remain less than $0.25\%$.

The measurements from DT3 require larger adjustments than those from DT4 and DT5 since the changes in the string angle for DT3, which occur due to the applied vertical displacements of the load block, are much larger than the changes in the string angles for DT4 and DT5, which occur due to the gap opening displacements at the beam ends. In evaluating the results from Test 3, adjusted measurements are used for $\Delta_{DT3}$; however, the measurements for $\Delta_{DT4}$ and $\Delta_{DT5}$ are not adjusted. Figure 6.164 plots $\Delta_{DT4}$, $\Delta_{DT5}$, and $\Delta_{DT3,x}$ against the beam chord rotation, $\theta_b$, respectively. It can be seen in Figures 6.164(a) and 6.164(b) that the vertical displacements of the load block at the north and south ends are nearly the same. The displacement in the $x$-direction is smaller under negative rotations, which is possibly due to any asymmetry in loading and/or behavior of the structure.

Combining these displacements, the $x$-displacement, $y$-displacement, and rotation of the load block centroid can be determined as described in Chapter 5 and shown in
Figures 6.165 and 6.166. Figure 6.165(a) plots the $y$-displacement versus the $x$-displacement showing the path of the load block centroid during the test. As the subassembly is displaced under positive (i.e., clockwise) and negative (i.e., counterclockwise) rotations, the load block is pushed north (in the $x$-direction, away from the reaction block) due to the gap opening at the beam ends. After each cycle, the load block returns to its initial position with minimal residual displacements. Figures 6.167(a) and 6.168(a) show that the $x$-direction displacement of the load block centroid is slightly smaller during the negative rotations of the subassembly as compared to the positive rotations, which is possibly due to any unsymmetric loading or behavior of the structure. In the $y$-direction, the load block displaces symmetrically during the positive and negative rotations as shown in Figures 6.166(a), 6.166(b), 6.167(d), and 6.168(b). Finally, the rotation of the load block is shown to remain small throughout the duration of the test as plotted in Figures 6.167(b) and 6.168(c). The load block rotation remains below 0.003 radians indicating that the two hydraulic actuators moved near simultaneously.
Test duration

Load Block Horizontal Displacement

1.3
(-15)
-0.6

displacement,
\( \Delta DT3 \) [in. (mm)]

(a)

Load Block Adjusted Horizontal Displacement

1.3
(-15)
-0.6

displacement,
\( \Delta DT3,x \) [in. (mm)]

(b)

Percent Difference

\( \text{abs}(\Delta DT3,x - \Delta DT3)/\Delta DT3 \) [per cent]

(c)

Figure 6.164: Test 3 load block horizontal displacements – (a) measured, \( \Delta DT3 \); (b) adjusted, \( \Delta DT3,x \); (c) percent difference.
Figure 6.165: Test 3 load block north end vertical displacements – (a) measured, $\Delta DT_4$; (b) adjusted, $\Delta DT_{4,y}$; (c) percent difference.
Figure 6.166: Test 3 load block south end vertical displacements – (a) measured, $\Delta DT5$; (b) adjusted, $\Delta DT5,y$; (c) percent difference.

Figure 6.167: Test 3 percent difference between measured and adjusted displacements – (a) DT4; (b) DT5.
Figure 6.168: Test 3 load block displacements versus beam chord rotation –
(a) $\Delta_{DT4}-\theta_b$; (b) $\Delta_{DT5}-\theta_b$; (c) $\Delta_{DT3,x}-\theta_b$. 
Figure 6.169: Test 3 load block centroid displacements – (a) x-y displacements; (b) rotation; (c) x-displacements; (d) y-displacements.
Figure 6.170: Test 3 load block centroid displacements versus beam chord rotation – (a) x-displacement-$\theta_b$; (b) y-displacement-$\theta_b$; (c) rotation-$\theta_b$.

6.4.11 Reaction Block Displacements and Rotations

String pots DT6 – DT8 are used to measure the vertical $y$-displacements and the horizontal $x$-displacement of the reaction block. Figure 6.169 shows the measurements from DT6 – DT8 for the duration of the test. Since the reaction block is tied to the strong floor, the measured displacements remain very small throughout the test. Due to these small displacements and the use of lead cables for each string pot, the change in angle that the string undergoes during testing is very small. Thus, it can be assumed that no
adjustments are needed for the displacements measured from the reaction block displacement transducers. Figure 6.170 plots the measurements from DT6 – DT8 against the beam chord rotation.

The $x$-displacement, $y$-displacement, and rotation of the reaction block centroid can be determined (see Chapter 5) as shown in Figure 6.171 and plotted against the beam chord rotation in Figure 6.172. It is concluded that the vertical displacements of the reaction block do not have a significant effect on the displacements of the test structure (e.g., the beam chord rotation), and the test results are presented with the reaction block displacements taken as zero (i.e., the measured displacements of the reaction block are ignored in investigating the response of the subassembly). Note that the horizontal displacements of the reaction block are significant when determining the total elongation of the post-tensioning tendon; and thus, are included in those calculations.
Figure 6.171: Test 3 reaction block displacements – (a) $\Delta DT_6$; (b) $\Delta DT_7$; (c) $\Delta DT_8$. 
Figure 6.172: Test 3 reaction block displacements versus beam chord rotation –
(a) $\Delta DT7-\theta_b$; (b) $\Delta DT8-\theta_b$; (c) $\Delta DT6-\theta_b$. 
Figure 6.173: Test 3 reaction block centroid displacements – (a) $x$-$y$ displacements; (b) rotation; (c) $x$-displacements; (d) $y$-displacements.
6.4.12 Contact Depth and Gap Opening at Beam-to-Wall Interfaces

The beam contact depth and gap opening displacements are measured at the beam-to-reaction-block interface using displacement transducers DT11 – DT13. As described in Chapter 5, these LVDTs rotate with the beam; and thus, their measurements may need to be adjusted to determine the gap opening displacements in the horizontal x-direction.
Figures 6.173 through 6.175 plot the measured displacements $\Delta DT11$, $\Delta DT12$, and $\Delta DT13$, respectively, the corresponding adjusted $x$-displacements $\Delta DT11,x$, $\Delta DT12,x$, and $\Delta DT13,x$, respectively, and the percent differences between the measured and adjusted displacements. The results indicate that the adjusted measurements are less than 0.15% different from the original measurements, and thus, $\Delta DT11$, $\Delta DT12$, and $\Delta DT13$ can be taken as the displacements in the $x$-direction. Figure 6.176 plots the measured data from DT11, DT12, and DT13 against the beam chord rotation.

The maximum average concrete compressive strain in the beam-to-wall contact regions can be calculated by dividing the measured displacements from DT11 and DT13 with the gauge length (i.e., the distance from the LVDT ferrule insert in the beam to the reaction plate ferrule insert in the wall test region; see Chapter 5). For Test 3, the maximum average compressive strain is 0.0036. Note that this measurement includes the compressive strain occurring in the fiber reinforced grout at the beam-to-wall interface and might have been affected by the damage occurring in the wall test region of the reaction block.
Figure 6.175: Test 3 beam-to-reaction-block interface top LVDT displacements – (a) measured, $\Delta DT11$; (b) adjusted, $\Delta DT11,x$; (c) percent difference.
Figure 6.176: Test 3 beam-to-reaction-block interface middle LVDT displacements – (a) measured, $\Delta DT12$; (b) adjusted, $\Delta DT12, x$; (c) percent difference.
Figure 6.177: Test 3 beam-to-reaction-block interface bottom LVDT displacements – (a) measured, $\Delta DT_{13}$; (b) adjusted, $\Delta DT_{13,x}$; (c) percent difference.
Using the measured data, the contact depth and the largest (i.e., at the beam top and bottom) gap opening displacements at the beam-to-reaction-block interface can be determined following the procedures in Chapter 5. Figures 6.177(a) and 6.178(a) show the results based on the measured data from DT11 – DT13 (method 1); Figures 6.177(b) and 6.178(b) show the results based on the measured data from RT1, DT11, and DT13 (method 2); Figures 6.177(c) and 6.178(c) show the results based on the measured data from RT1 and DT12 (method 3); Figures 6.177(d) and 6.178(d) show the results based on the beam chord rotation, $\theta_b$, and the measurements from DT12 (method 4); and Figures
6.177(e) and 6.178(e) show the results based on the beam chord rotation, $\theta_b$ and the measurements from DT11 and DT13 (method 5). Each ○ marker in Figures 6.177 and 6.178 indicates the contact depth or gap opening displacement at the peak of a loading cycle up to a beam chord rotation of 5.0%.

Looking at Figure 6.177, it can be stated that the contact depth results obtained using the five methods are somewhat different but show similar trends. There is a rapid reduction in the contact depth up to a beam chord rotation of about 2.0%. After this rotation, the contact depth remains relatively stable due to nonlinear behavior of the concrete in compression and varies between 4.0 – 1.0 in. (102 – 51 mm) [about 30 – 7.0% of the beam depth]. In comparison, the gap opening results using the five methods in Figure 6.178 are reasonably similar and the increase in gap opening with the rotation of the beam is very close to linear.

Figure 6.179 shows a continuous plot of the largest gap opening displacements determined from method 4 (using the beam chord rotation and DT12) against the beam chord rotation. Similarly, Figure 6.180(a) plots the beam contact depth from method 4 against the beam chord rotation as continuous data. Furthermore, Figures 6.180(b) and 6.180(c) show continuous plots of the contact depth during the 2.25% and 5.0% beam chord rotation cycles, respectively. It can be seen that unlike Tests 1 and 2, the contact depth behavior during the 2.25% cycle in Test 3 is similar to the behavior during the 5.0% cycle. It is unclear if the instruments and/or beam shifted to skew the data or if this is the actual behavior of the beam. Furthermore, the gap opening and contact depth plots in Figures 6.167 and 6.168 could have been affected by the use of $\theta_b$ (i.e., chord rotation) instead of $\theta_{RT1}$ (i.e., local rotation) to determine the behavior at the south end of the beam.
The use of the beam midheight transducer DT12 instead of the extreme beam top or bottom transducer (DT11 or DT13) may also have affected the results, especially the contact depth plots in Figure 6.168 during small beam rotations, which are very sensitive to the measurements.

In addition to the LVDT measurements, the gap opening measurements at the south end of the beam were taken using a ruler. These ruler measurements are listed in Table 6.3 and shown using + markers in Figure 6.178. It can be seen that the LVDT gap opening measurements (○ markers) in Figure 6.178 are reasonably close to the ruler measurements.
Figure 6.179: Test 3 contact depth at beam-to-reaction-block interface –
(a) method 1 using DT11, DT12, and DT13; (b) method 2 using RT1, DT11, and DT13; (c) method 3 using RT1 and DT12; (d) method 4 using $\theta_b$ and DT12; (e) method 5 using $\theta_b$, DT11, and DT13.
Figure 6.180: Test 3 gap opening at beam-to-reaction-block interface –
(a) method 1 using DT11, DT12, and DT13; (b) method 2 using RT1, DT11, and DT13;
(c) method 3 using RT1 and DT12; (d) method 4 using $\theta_b$ and DT12; (e) method 5 using $\theta_b$, DT11, and DT13.
Figure 6.181: Test 3 gap opening at beam-to-reaction-block interface using method 4.

Figure 6.182: Test 3 contact depth at beam-to-reaction-block interface using method 4 – (a) entire data; (b) 2.25% beam chord rotation cycle; (c) 5.0% beam chord rotation cycle.
6.4.13 Wall Test Region Local Concrete Deformations

The reaction block confined concrete deformations near the beam-to-wall interface of the wall test region are measured using displacement transducers DT14 and DT15. Figure 6.181 plots the time history results from the top (DT14) and bottom (DT15) transducers, and Figure 6.182 plots the measured data from DT14 and DT15 against the beam chord rotation. As expected, the concrete deformations are mostly compressive (negative) due to the compression stresses that are transferred through the contact region from post-tensioning.

From Figure 6.181, the maximum average concrete compressive strain in the wall test region can be calculated by dividing the measured displacements with the gauge length (i.e., the distance from the LVDT ferrule insert in the wall test region to the
reaction plate; see Chapter 5). For Test 3, the maximum average concrete compressive strain is 0.004, which is equal to the expected unconfined (cover) concrete crushing strain of 0.004. This finding is in accordance with the visual observation that concrete spalling occurred in the wall test region during the test.

![Reaction Block LVDT Displacement](image1)

(a)

![Reaction Block LVDT Displacement](image2)

(b)

Figure 6.183: Test 3 wall test region concrete deformations – (a) DT14; (b) DT15.

![Displacement vs Beam Chord Rotation](image3)

(a)

![Displacement vs Beam Chord Rotation](image4)

(b)

Figure 6.184: Test 3 wall test region concrete deformations versus beam chord rotation – (a) $\Delta DT14 - \theta_b$; (b) $\Delta DT15 - \theta_b$. 

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6.4.14 Beam Looping Reinforcement Longitudinal Leg Strains

Figures 6.183 and 6.184 show the strain gauge measurements for the top and bottom horizontal legs of the east and west No. 6 mild steel looping reinforcing bars in the beam. The locations of these strain gauges can be found in Chapter 5. The initial beam post-tensioning force results in a small compressive strain in the bars at the beginning of the test. Upon lateral loading of the subassembly, the largest tensile strains occur, as expected, in the gauges closest to the angle-to-beam connection bolts [i.e., gauges 6(1)T-E, 6(1)T-W, 6(1)B-E, and 6(1)B-W]. The measurements in the gauges away from the angle-to-beam connection decrease with distance from this critical location.

To provide a better understanding of the strain measurements in the horizontal legs of the beam looping reinforcement, the \( \Delta \) and \( \Box \) markers in Figures 6.183(a) and 6.183(b) for gauges 6(1)T-E and 6(1)T-W correspond to positive and negative chord rotation peaks for the beam, respectively, and the \( \circ \) markers indicate zero rotation positions. To provide further insight into the results, Figures 6.185 and 6.186 show the strains plotted against the beam chord rotation. In the positive (i.e., clockwise) rotation direction, the strains in the top bars increase in tension as the gap opens at the top south corner of the beam and the top angle is pulled in tension. In the negative (i.e., counterclockwise) direction, the strains in the bottom bars increase in tension and the top bars go into compression due to the closing of the gap.

The maximum strains in the four gauges closest the critical section (i.e., the angle-to-beam connection) remain well below the yield strain of the longitudinal steel (\( \varepsilon_{y} = \))
0.00283) from the material tests in Chapter 4; and thus, it is concluded that the amount of mild steel reinforcement used to transfer the angle forces into the beam is adequate.

Figure 6.185: Test 3 beam looping reinforcement top longitudinal leg strains – (a) $\varepsilon_{6(1)T-E}$; (b) $\varepsilon_{6(1)T-W}$; (c) $\varepsilon_{6(2)T-E}$; (d) $\varepsilon_{6(2)T-W}$; (e) $\varepsilon_{6(3)T-E}$; (f) $\varepsilon_{6(3)T-W}$; (g) $\varepsilon_{6MT-E}$; (h) $\varepsilon_{6MT-W}$. 
Figure 6.185 continued.

Figure 6.186: Test 3 beam looping reinforcement bottom longitudinal leg strains –
(a) $\varepsilon_{(1)B-E}$; (b) $\varepsilon_{(1)B-W}$; (c) $\varepsilon_{(2)B-E}$; (d) $\varepsilon_{(2)B-W}$; (e) $\varepsilon_{(3)B-E}$; (f) $\varepsilon_{(3)B-W}$; (g) $\varepsilon_{6MB-E}$; (h) $\varepsilon_{6MB-W}$.
Figure 6.186 continued.
Figure 6.187: Test 3 beam looping reinforcement top longitudinal leg strains versus beam chord rotation –
(a) $\varepsilon_{6(1)T-E} - \theta_b$; (b) $\varepsilon_{6(1)T-W} - \theta_b$; (c) $\varepsilon_{6(2)T-E} - \theta_b$;
(d) $\varepsilon_{6(2)T-W} - \theta_b$; (e) $\varepsilon_{6(3)T-E} - \theta_b$; (f) $\varepsilon_{6(3)T-W} - \theta_b$; (g) $\varepsilon_{6MT-E} - \theta_b$; (h) $\varepsilon_{6MT-W} - \theta_b$. 

(no data collected from strain gauge 6(2)T-W)
no data collected from strain gauge 6(3)T-W

Figure 6.187 continued.
Figure 6.188: Test 3 beam looping reinforcement bottom longitudinal leg strains versus beam chord rotation – (a) $\varepsilon_{6(1)B-E} - \theta_b$; (b) $\varepsilon_{6(1)B-W} - \theta_b$; (c) $\varepsilon_{6(2)B-E} - \theta_b$; (d) $\varepsilon_{6(2)B-W} - \theta_b$; (e) $\varepsilon_{6(3)B-E} - \theta_b$; (f) $\varepsilon_{6(3)B-W} - \theta_b$; (g) $\varepsilon_{6MB-E} - \theta_b$; (h) $\varepsilon_{6MB-W} - \theta_b$. 

no data collected from strain gauge
$6(2)B-E$

no data collected from strain gauge
$6(2)B-W$
6.4.15 Beam Transverse Reinforcement Strains

Figure 6.187 shows the strain measurements from gauges 6SE(I)-E, 6SE(E)-E, 6SE(I)-W, and 6SE(E)-W placed on the transverse (i.e., vertical) legs of the east and west No. 6 looping reinforcing bars at the south end of the beam. The locations of these strain gauges can be found in Chapter 5. To give more insight into the measurements, Figure 6.188 plots the strain data against the beam chord rotation. It can be seen that the strain gauge readings remain well below the yield strain of the reinforcing steel ($\varepsilon_{\text{y}} = 0.00283$, see Chapter 4) throughout the experiment, demonstrating that the design of the transverse reinforcement at the beam ends is adequate. Note that the angle-to-beam connection bolts may also have acted as transverse reinforcement in the beam; however, this could not be confirmed from the test results since the connection bolts were not instrumented.

Note that no strain measurements were collected from gauges MH-E and MH-W placed on the vertical legs of the No. 3 transverse hoop at the beam midspan (Figures 6.189 and 6.190). However, the strain results from Tests 1 and 2, and visual observations
from Test 3 indicate that the use of nominal transverse reinforcement within the span of
the beam is adequate.

Figure 6.189: Test 3 beam looping reinforcement vertical leg strains –
(a) $\varepsilon_{6SE(E)-E}$; (b) $\varepsilon_{6SE(E)-W}$; (c) $\varepsilon_{6SE(I)-E}$; (d) $\varepsilon_{6SE(I)-W}$. 
strain gauge 6SE (E)·E removed from Beam #3
strain gauge 6SE (E)·W removed from Beam #3

(a) (b)
beam chord rotation, θb (%)
strain, ε
6SE(I)·E
0.0008 -0.0001
0 0
6-6 0
345

(c) (d)
beam chord rotation, θb (%)
strain, ε
6SE(I)·W
0.0008 -0.0001
0 0
360

Figure 6.190: Test 3 beam looping reinforcement vertical leg strains versus beam chord rotation – (a) ε6SE(E)·E·θb; (b) ε6SE(E)·W·θb; (c) ε6SE(I)·E·θb; (d) ε6SE(I)·W·θb.

no data collected from strain gauge MH-E
no data collected from strain gauge MH-W

(a) (b)

Figure 6.191: Test 3 beam midspan transverse hoop reinforcement strains – (a) εMH·E; (b) εMH·W.
no data collected from strain gauge
MH-E

no data collected from strain gauge
MH-W

(a) (b)

Figure 6.192: Test 3 beam midspan transverse hoop reinforcement strains versus beam chord rotation – (a) $\varepsilon_{\text{MH-E}} - \theta_b$; (b) $\varepsilon_{\text{MH-W}} - \theta_b$.

6.4.16 Beam Confined Concrete Strains

Figures 6.191 and 6.192 show the measurements from the strain gauges placed on the No. 3 support bars inside the hoop confined concrete at the south end of the beam. The locations of these strain gauges can be found in Chapter 5. For further insight into the strain gauge readings, Figures 6.193 and 6.194 plot the strain data against the beam chord rotation. The results show that there is a small compressive strain at the beginning of the test due to the initial post-tensioning force. As the beam is rotated in the positive (i.e., clockwise) direction, the compression strains in the bottom bars increase due to the transfer of the contact stresses through the bottom corner of the beam. In the opposite (i.e., counterclockwise) direction, the strains in the top and bottom bars reverse due to the reversal of the load. It can be seen that the strain gauge measurements from the support bars remain below the steel yield strain in tension and below the confined concrete crushing strain in compression.
strain gauge 3THT-(1) removed from Beam 3

strain gauge 3THB-(1) removed from Beam 3

strain gauge 3THT-(2) removed from Beam 3

strain gauge 3THB-(2) removed from Beam 3

Figure 6.193: Test 3 No. 3 top hoop support bar strains—
(a) $\varepsilon_{3THT-(1)}$; (b) $\varepsilon_{3THB-(1)}$; (c) $\varepsilon_{3THT-(2)}$; (d) $\varepsilon_{3THB-(2)}$. 

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Figure 6.194: Test 3 No. 3 bottom hoop support bar strains—
(a) $\varepsilon_{3BHB-(1)}$; (b) $\varepsilon_{3BHT-(1)}$; (c) $\varepsilon_{3BHB-(2)}$; (d) $\varepsilon_{3BHT-(2)}$. 

strain gauge 3BHT-(1) removed from Beam 3

strain gauge 3BHT-(2) removed from Beam 3
strain gauge 3THT-(1) removed from Beam 3
strain gauge 3THB-(1) removed from Beam 3

(a) (b)
strain gauge 3THT-(2) removed from Beam 3
strain gauge 3THB-(2) removed from Beam 3

(c) (d)

Figure 6.195: Test 3 No. 3 top hoop support bar strains versus beam chord rotation –
(a) $\varepsilon_{3THT-(1)} - \theta_b$; (b) $\varepsilon_{3THB-(1)} - \theta_b$; (c) $\varepsilon_{3THT-(2)} - \theta_b$; (d) $\varepsilon_{3THB-(2)} - \theta_b$. 

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Figure 6.196: Test 3 No. 3 bottom hoop support bar strains versus beam chord rotation – (a) $\varepsilon_{3BHB-(1)} - \theta_b$; (b) $\varepsilon_{3BHT-(1)} - \theta_b$; (c) $\varepsilon_{3BHB-(2)} - \theta_b$; (d) $\varepsilon_{3BHT-(2)} - \theta_b$.

6.4.17 Beam End Confinement Hoop Strains

Figures 6.195 and 6.196 show the measurements from the strain gauges placed on the vertical legs of the bottom layer No. 3 confinement hoops at the south end of the beam. The locations of these strain gauges can be found in Chapter 5. For further insight into the measurements, Figures 6.197 and 6.198 plot the strain data against the beam chord rotation. The angle-to-beam connection bolt forces result in a small compressive strain in the vertical hoop steel at the beginning of the test, with the exception of 3HB-W.
Unlike Tests 1 and 2, the beam end confinement hoop strains in Test 3 are tensile. It can be seen that throughout the test, the measured confinement hoop strains remain small.

Figure 6.197: Test 3 beam end confinement hoop east leg strains – (a) $\varepsilon_{1HBE}$; (b) $\varepsilon_{2HBE}$; (c) $\varepsilon_{3HBE}$; (d) $\varepsilon_{4HBE}$. 
Figure 6.198: Test 3 beam end confinement hoop west leg strains –
(a) $\varepsilon_{1HBW}$; (b) $\varepsilon_{2HBW}$; (c) $\varepsilon_{3HBW}$; (d) $\varepsilon_{4HBW}$. 

strain gauge 2HB-W removed from Beam 3

strain gauge 4HB-W removed from Beam 3
Figure 6.199: Test 3 beam end confinement hoop east leg strains versus beam chord rotation – (a) $\varepsilon_{1HB-E}-\theta_b$; (b) $\varepsilon_{2HB-E}-\theta_b$; (c) $\varepsilon_{3HB-E}-\theta_b$; (d) $\varepsilon_{4HB-E}-\theta_b$. 

(c) 

no data collected from strain gauge 3HB-E
Figure 6.200: Test 3 beam end confinement hoop west leg strains versus beam chord rotation – (a) $\varepsilon_{1HBW}$-\(\theta_b\); (b) $\varepsilon_{2HBW}$-\(\theta_b\); (c) $\varepsilon_{3HBW}$-\(\theta_b\); (d) $\varepsilon_{4HBW}$-\(\theta_b\).

### 6.4.18 Wall Test Region Confined Concrete Strains

As described previously, the strain gauge wires coming out of the reaction block were all severed during the removal of the steel casting mold. Thus, no measurements were recorded for the wall test wall region confined concrete strains.

### 6.4.19 Wall Test Region Confinement Hoops Strains

Similar to above, no measurements were recorded for the wall test region confinement hoop strains since the strain gauge wires were severed.
6.4.20 Crack Patterns

The crack propagation from testing is marked on the beam during each cycle of displacement (e.g., see Figures 6.140, 6.141, and 6.142). In addition, the crack propagation is recorded by approximately copying the crack patterns manually onto paper. Figure 6.201 shows the hand-drawn crack patterns recorded for Test 3 at the end of the last cycle from selected displacement increments (see Appendix F for crack sheets).

Figure 6.201: Test 3 crack patterns – (a) $\theta_b = 0.125\%$; (b) $\theta_b = 0.175\%$; (c) $\theta_b = 0.25\%$; (d) $\theta_b = 0.35\%$; (e) $\theta_b = 0.5\%$; (f) $\theta_b = 0.75\%$; (g) $\theta_b = 1.0\%$; (h) $\theta_b = 1.5\%$; (i) $\theta_b = 2.25\%$; (j) $\theta_b = 3.33\%$; (k) $\theta_b = 5.0\%$. 

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Figure 6.201 continued.
6.5 Test 4

The beam used in Test 4 (i.e., Beam 4) has the following properties: (1) beam depth, \( h_b = 18 \text{ in. (457 mm)} \); (2) beam width, \( b_b = 7.5 \text{ in. (191 mm)} \); (3) mild steel reinforcement of two No. 6 bars looping around the beam perimeter along its length; (4) No. 3 full-depth rectangular hoops [6.125 in. by 12.675 in. (156 mm by 322 mm)] placed at a nominal 7.0 in. (178 mm) spacing to provide transverse reinforcement in the beam midspan region; (5) No. 3 partial-depth rectangular hoops [6.125 in. by 4.375 in. (156 mm by 111 mm)] placed at a 1.5 in. (38 mm) spacing to provide concrete confinement at the beam ends; (6) a beam post-tensioning tendon comprised of three 0.6 in. (15 mm) nominal diameter high-strength strands with a total area of \( A_{bp} = 0.651 \text{ in.}^2 \) (420 mm\(^2\)); (7) average initial beam post-tensioning strand stress of \( f_{bpi} = 0.43 f_{bpu} \), where \( f_{bpu} = 270 \) ksi (1862 MPa) is the design maximum strength of the post-tensioning steel; (8) total initial beam post-tensioning force of \( P_{bi} = 76.3 \text{ kips (340 kN)} \); (9) initial beam concrete nominal axial stress (based on actual cross-sectional area with beam post-tensioning duct removed) of \( f_{bc} = 0.58 \text{ ksi (4.0 MPa)} \); and (10) two top and two seat angles (L8x8x1/2) comprised of 2 – 2.5 in. (64 mm) long angle strips resulting in a total angle length of \( l_a = 5.0 \text{ in. (127 mm)} \).

The primary parameter differences of Test 4 from Test 3 are: (1) increased beam depth; (2) use of angle-to-wall connection plates (as described later); and (3) reduced initial beam concrete nominal axial stress. The same displacement loading history from Tests 2 and 3 was used for Test 4. Furthermore, the measures taken to prevent premature wire fractures in the beam post-tensioning strands in Tests 2 and 3 were also used in Test 4, as well as the use of a reduced height grout column at the beam-to-wall interfaces.
6.5.1 Test Photographs

Photographs of the original and displaced subassembly configurations from Test 4 are shown in Figures 6.202 and 6.203. Figures 6.202(a) through 6.202(f) show overall subassembly photographs as follows: (a) pre-test undisplaced position; (b) displaced to $\theta_b = 3.33\%$; (c) displaced to $\theta_b = -3.33\%$; and (d) final post-test undisplaced position. Similarly, Figures 6.203(a) through 6.203(f) show close-up photographs of the south end of the beam at the beam-to-reaction-block interface. The accumulation of damage at the south end of the beam is shown in more detail in Figure 6.204.

As described previously, the use of two short angle strips in Test 3 led to damage in the wall test region of the reaction block. Angle-to-wall connection plates were placed behind the vertical legs of the angles to limit additional damage occurring in the wall test region during the subsequent tests utilizing short angle strips [instead of full 7.5 in. (191 mm) long angles]. As shown in Figure 6.205, each angle-to-wall connection plate is 0.5 in. (13 mm) thick, has a width equal to the 7.5 in. (191 mm) width of the wall test region, and is 7.5 in. (191 mm) in height. A small gap was left between each plate and the corner of the beam in order to prevent the heel of the angle from coming into contact with the plate during large rotation cycles.

The wall test region of the reaction block did not receive any additional damage (no cracking and/or spalling of the cover concrete) during Test 4 after being patched with a high strength fiber reinforced grout following the Test 3 series (see Figure 6.206). Minimal spalling of the cover concrete at the beam ends was seen throughout the duration of the test. The test was stopped at a beam chord rotation of 3.33%, so that the beam could be retested with additional parameter variations as described in Chapter 7.
The angle-to-wall connections performed well with no yielding in the connection strands and no additional damage to the wall concrete. Slip between the coupling beam and the walls did not occur demonstrating that the friction resistance due to the post-tensioning force provided adequate vertical support to the beam together with resistance from the top and seat angles. Similar to Tests 2 and 3, premature wire fracture in the post-tensioning strands did not occur in Test 4. Furthermore, similar to Test 3, no slip in the angle-to-beam connections was observed during the test (unlike Tests 1 and 2), possibly due to the reduced amount of damage to the concrete at the beam ends.

Under smaller displacements of the coupling beam up to about 2.25% rotation, gap opening occurred between the fiber-reinforced grout and the faces of the load and reaction blocks. Upon further displacements of the structure, gap opening continued to form until the end of the test; however, it was observed that gaps formed on both sides of the grout column. While this is not the desired mode of behavior for the structure as described in Chapter 3, it did not change the behavior of the system or cause any problems with the performance of the grout. No significant crushing of the grout at the beam-to-wall interfaces was observed throughout the test. Note that the grout column was left approximately 0.25 in. (6.4 mm) short of the beam depth at the top and bottom, as was also done in Tests 2 and 3.
Figure 6.202: Test 4 overall photographs –
(a) pre-test undisplaced position; (b) $\theta_b = 3.33\%$; (c) $\theta_b = -3.33\%$;
(d) final post-test undisplaced position.
Figure 6.203: Test 4 beam south end photographs –
(a) pre-test undisplaced position; (b) $\theta_b = 3.33\%$; (c) $\theta_b = -3.33\%$;
(d) final post-test undisplaced position.
\[ \theta_b = 0.5\% \ 1.0 \ 1.5 \ 2.25 \ 3.33 \]

\[ \theta_b = -0.5\% \ -1.0 \ -1.5 \ -2.25 \ -3.33 \]

Figure 6.204: Test 4 beam south end damage propagation – positive and negative rotations.

Figure 6.205: Test 4 angle-to-wall connection plates.
6.5.2 Beam Shear Force versus Chord Rotation Behavior

Figure 6.207(a) shows the hysteretic coupling beam shear force, $V_b$ versus chord rotation, $\theta_b$ behavior from Test 4, where $V_b$ and $\theta_b$ are calculated from Equations 5.5 and 5.35, respectively. For comparison, Figure 6.207(b) plots the $V_b$-$\theta_{b,lb}$ behavior, where $\theta_{b,lb}$ is the beam chord rotation determined from the vertical ($y$-direction) displacement of the load block centroid using Equation 5.36. As shown in Figure 6.207, the structure was able to sustain three cycles at 3.33% rotation with no loss in shear resistance. The test was stopped at this point (prior to the failure of the beam) to prevent any further damage to the beam so that it could be reused for additional parametric investigations.

Looking at the hysteresis loops, it can be seen that the specimen was able to dissipate a considerable amount of energy. Most of this energy dissipation occurred due
to the yielding of the top and seat angles. The post-tensioning tendon did not have any wire fracture during the test due to the reduced initial stress in the post-tensioning strands and the use of additional anchor barrels to reduce strand “kinking” inside the anchors (see Section 6.5.4). The beam had a sufficient amount of restoring force to yield the tension angles back in compression and close the gaps at the beam-to-wall interfaces upon unloading, resulting in a large self-centering capability.
Figure 6.207: Test 4 coupling beam shear force versus chord rotation behavior –
(a) using beam displacements; (b) using load block displacements.
6.5.3 Beam End Moment Force versus Chord Rotation Behavior

Figure 6.208 shows the hysteretic coupling beam end moment, $M_b$, versus chord rotation, $\theta_b$, behavior from Test 4, where $M_b$ is calculated from Equation 5.6 as described in Chapter 5. Since the beam end moment is calculated from the beam shear force, the results shown in Figure 6.208 are directly related to the results in Figure 6.207; and thus, no further discussion is provided herein.

![Figure 6.208: Test 4 $M_b$-$\theta_b$ behavior.](image-url)
6.5.4 Beam Post-Tensioning Forces

The coupling beam post-tensioning forces from the test are measured using load cells LC15 – LC17 mounted at the dead ends of the three strands (see Chapter 5). The forces from the three load cells, $F_{LC15}$, $F_{LC16}$, and $F_{LC17}$ are shown in Figure 6.209, and are plotted against the beam chord rotation, $\theta_b$ in Figure 6.210. Figure 6.211 shows the total beam post-tensioning tendon force, $P_{bp}$ (sum of the forces in the three strands) normalized with the total design ultimate strength of the tendon, $\Sigma a_{bp}f_{bpu}$. The total initial beam post-tensioning tendon force is equal to 76.3 kips (340 kN), resulting in an initial beam nominal concrete axial stress of $f_{bci} = 0.58$ ksi (4.0 MPa).

As the structure is displaced and gap opening occurs at the beam ends, the post-tensioning strands elongate and the post-tensioning forces increase. Since the strands are unbonded over the entire length of the subassembly, the nonlinear straining of the post-tensioning steel is significantly delayed, with only small losses occurring in the post-tensioning forces (possibly due to small amounts of nonlinear straining of the post-tensioning steel, additional seating of the anchor wedges, and/or nonlinear behavior in the beam/wall concrete).

Note that similar steps were taken in Test 4 as in Tests 2 and 3 to help prevent premature wire fractures in the post-tensioning strands, including the use of reduced initial post-tensioning stresses and the use of extra anchor barrels to reduce strand “kinking” inside the anchor wedges (see Section 6.3.4). These measures were successful to achieve satisfactory performance of the strand/anchor system during the experiment.
Figure 6.209: Test 4 beam post-tensioning strand forces – (a) strand 1; (b) strand 2; (c) strand 3.
Figure 6.210: Test 4 $F_{LC}-\theta_b$ behavior – (a) strand 1; (b) strand 2; (c) strand 3.
Figure 6.211: Test 4 beam post-tensioning force versus chord rotation behavior – (a) using beam displacements; (b) using load block displacements.
6.5.5 Angle-to-Wall Connection Post-Tensioning Forces

The beam south end (i.e., reaction block end) angle-to-wall connection post-tensioning strand forces, $F_{LC3} - F_{LC6}$, measured using load cells LC3 – LC6, respectively, are shown in Figures 6.212 through 6.215. The target initial force for each connection strand is 20 kips (89 kN); whereas, the measured initial forces in the four strands are $F_{i,LC3} = 27.0$ kips (120 kN), $F_{i,LC4} = 27.4$ kips (122 kN), $F_{i,LC5} = 30.8$ kips (137 kN), and $F_{i,LC6} = 25.7$ kips (114 kN). A considerable variation is observed in the initial connection strand forces since even a slight difference in the amount of anchor wedge seating has a large effect on the initial force (due to the short length of the strands).

Figures 6.216 (a) and 6.216(b) show the total forces in the south top and south seat angle connection strands, respectively, plotted against the beam chord rotation, $\theta_b$. The connection forces are normalized with the total design ultimate strength of the strands, $P_{abu} = \sum a_{ap} f_{apu}$, where $f_{apu} = 270$ ksi (1862 MPa). The expected behavior of the strands is that as the structure is displaced and the angles are pulled in tension, the connection forces increase; and upon unloading, the connection forces return more or less back to the initial forces with possibly some losses occurring due to additional seating of the anchor wedges and any permanent deformations in the concrete (note that the nonlinear straining of the post-tensioning steel is prevented since the strands are left unbonded). These trends are seen in load cell LC5; however, the forces from the other load cells do not behave as expected. This could have been due to the malfunctioning of the load cells under the non-uniform loads applied during the prying deformations of the angles.
Figure 6.212: Test 4 south end top angle-to-wall connection strand forces – (a) east strand; (b) west strand.

Figure 6.213: Test 4 south end top angle-to-wall connection strand forces versus beam chord rotation – (a) $F_{LC3}-\theta_b$; (b) $F_{LC4}-\theta_b$. 

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Figure 6.214: Test 4 south end seat angle-to-wall connection strand forces – (a) east strand; (b) west strand.

Figure 6.215: Test 4 south end seat angle-to-wall connection strand forces versus beam chord rotation – (a) $F_{LC5}-\theta_b$; (b) $F_{LC6}-\theta_b$. 
6.5.6 Vertical Forces on Wall Test Region

Load cells LC7 – LC14 are used to measure the forces in the eight vertical bars applying axial compression forces to the wall test region of the reaction block and anchoring the block to the strong floor. The total vertical force, $F_{wt}$, is determined as described in Chapter 5, with the target initial total force ranging between 150 – 160 kips (667 – 712 kN). Figure 6.217(a) shows $F_{wt}$ for the duration of the test and Figure 6.217(b) plots $F_{wt}$ against the beam chord rotation, $\theta_b$. The initial total force, $F_{wt,i}$, is 148 kips (658 kN), slightly below the target force range. As the beam is rotated in the positive (i.e., clockwise) direction with the load block moving down, $F_{wt}$ decreases since the beam applies a downward force on the reaction block. Similarly, as the beam is rotated in the negative (i.e., counterclockwise) direction, $F_{wt}$ increases since the beam applies an upward force on the reaction block. Note that, as described in Chapter 5, the amount of variation in $F_{wt}$ during the cyclic displacements of the beam is relatively small as
compared with the expected variation of axial forces in the wall pier coupling regions of a multi-story coupled wall system. Upon unloading, $F_{w_t}$ returns more or less to its initial value, with some amount of loss occurring possibly due to the loosening of the nuts on the bars and/or any permanent deformations in the reaction block concrete.

![Figure 6.217: Test 4 vertical force on the wall test region, $F_{w_t}$ – (a) $F_{w_t}$-test duration; (b) $F_{w_t}$-$\theta_b$](image)

**6.5.7 Beam Vertical Displacements**

The vertical displacements $\Delta_{DT9}$ and $\Delta_{DT10}$ at the south and north ends of the beam are measured using string pots DT9 and DT10, respectively. These displacements are used to calculate the beam chord rotation, $\theta_b$. As described in Chapter 5, as the subassembly is displaced, the transducer string undergoes a change of angle, which can be “adjusted” to give the vertical displacements in the $y$-direction. Figures 6.218 and 6.219 show the measured displacements $\Delta_{DT9}$ and $\Delta_{DT10}$, respectively, the corresponding adjusted $y$-displacements $\Delta_{DT9,y}$ and $\Delta_{DT10,y}$, respectively, and the difference between the measured and adjusted displacements for the duration of the test. Note that there is a
downward drift in the data from DT9 starting at approximately 1.0% beam chord rotation indicating the beam might have shifted in the vertical direction with respect to the wall; however, it is not clear if this was due to slip and/or repositioning of the beam since no visual observations were made.

It can be seen from Figures 6.218 and 6.219 that $\Delta_{DT9}$ and $\Delta_{DT10}$ are very close to $\Delta_{DT9,y}$ and $\Delta_{DT10,y}$, respectively, with the difference being less than 0.005 in. (0.13 mm). Figure 6.220 plots the percent difference between the measured and adjusted displacements versus the beam chord rotation, $\theta_b$. The results indicate that the largest percent differences occur when the beam chord rotation is close to zero; however, these differences are not significant since the corresponding measurements are very small and are mostly outside the sensitivity of the transducers. As the structure is displaced, the measurements from DT9 require larger adjustments than those from DT10, since corresponding to a given $\theta_b$, $\Delta_{DT9}$ is smaller than $\Delta_{DT10}$. It is also observed that for negative rotations, DT9 displays larger percent errors than under positive rotations because the measured displacements under negative rotations are smaller (possibly due to the drift in the data) than the measured displacements under positive rotations. Therefore, similar differences between the measured and adjusted displacements under the negative and positive directions result in larger percent errors in the negative direction. Since the adjustments described in Chapter 5 require certain assumptions and approximations and since the amplitude differences (which are more important than percent differences for the calculations of the beam chord rotation) between the measured and adjusted displacements remain small, these differences are ignored and the measurements from DT9 and DT10 are used as the vertical $y$-displacements of the beam throughout this
dissertation. Figure 6.221 plots the measured data from DT9 and DT10 against the beam chord rotation, $\theta_b$. 

![Figure 6.218: Test 4 south end beam vertical displacements – (a) measured, $\Delta_{DT9}$; (b) adjusted, $\Delta_{DT9,y}$; (c) difference, $\Delta_{DT9,y} - \Delta_{DT9}$.](image-url)
Figure 6.219: Test 4 north end beam vertical displacements – (a) measured, $\Delta_{DT10}$; (b) adjusted, $\Delta_{DT10,y}$; (c) difference, $\Delta_{DT10,y} - \Delta_{DT10}$.

Figure 6.220: Test 4 percent difference between measured and adjusted displacements – (a) south end, DT9; (b) north end, DT10.
6.5.8 Beam Chord Rotation

The beam chord rotation is defined as the relative vertical displacement of the beam ends divided by the beam length. The beam chord rotation $\theta_b$ determined based on the $\Delta DT9$ and $\Delta DT10$ measurements in Test 4 is shown in Figure 6.222(a). For comparison, Figure 6.222(b) shows the load block beam chord rotation, $\theta_{b,lb}$ calculated using $\Delta LB,y$, and Figure 6.223 shows the percent difference between $\theta_b$ and $\theta_{b,lb}$ plotted against $\theta_b$. It can be seen that there is a large percent difference for small beam chord rotations, with the difference dropping down to less than 10% at larger beam rotations. Figure 6.224 plots the beam chord rotation, $\theta_b$ and the load block beam chord rotation, $\theta_{b,lb}$ against the beam chord rotation, $\theta_b$. The results show that the two chord rotations are approximately equal under negative loading, but have a small difference under positive loading with $\theta_{b,lb}$ being slightly larger.
Figure 6.222: Test 4 beam chord rotation –
(a) $\theta_b$ from $\Delta_{DT9}$ and $\Delta_{DT10}$; (b) $\theta_{b,lb}$ from $\Delta_{LB,y}$.

Figure 6.223: Test 4 percent difference between $\theta_b$ and $\theta_{b,lb}$. 

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6.5.9 Local Beam Rotations

Local beam rotations were measured using two rotation transducers (inclinometers); one near the midspan of the beam (RT1) and the other near the south end (RT2). Figure 6.225 shows the time rotation history results, $\theta_{RT1}$ and $\theta_{RT2}$ from these two transducers. Both rotation measurements are positive when the load block is displaced in the downward direction (i.e., clockwise beam rotation). The beam south end rotations from RT1 starts to drift from the origin very early during the test, possibly due to a malfunction; and thus, when compared to the beam chord rotation as shown in Figure 6.226(a), there is a large difference between the two rotations. For the beam midspan, Figure 6.226(b) shows that the rotations from RT2 are larger than the beam chord rotations. Note that due to the flexural deformations of the beam, it would be expected that the beam chord rotation, $\theta_b$ is smaller than $\theta_{RT2}$ but larger than $\theta_{RT1}$ [see Figure 6.224: Test 4 difference between $\theta_b$ and $\theta_{b,lb}$.]
6.226(c)]. This behavior is not observed in Figure 6.226(a) for $\theta_{RT1}$ in the negative direction.

Figure 6.225: Test 4 beam inclinometer rotations – (a) near beam south end, $\theta_{RT1}$; (b) near beam midspan, $\theta_{RT2}$. 
Figure 6.226: Test 4 difference between beam inclinometer rotations and beam chord rotations – (a) RT1; (b) RT2; (c) beam deflected shape.

6.5.10 Load Block Displacements and Rotations

String pots DT3 – DT5 are used to measure the vertical $y$-displacements and the horizontal $x$-displacement of the load block. Similar to the vertical beam displacements, as the load block is displaced, the strings of the load block displacement transducers undergo a change in angle; and thus, their measurements may need to be adjusted to give the $x$- and $y$-displacements of the load block as described in Chapter 5.

Figures 6.227 through 6.229 show the measured displacements $\Delta DT3$, $\Delta DT4$, and $\Delta DT5$, respectively, the corresponding adjusted displacements $\Delta DT3,x$, $\Delta DT4,y$, and $\Delta DT5,y$, respectively, and the percent differences between the measured and adjusted
displacements. Note that, as described in Chapter 5, the negative $\Delta_{DT3}$ and $\Delta_{DT3,x}$ measurements indicate the movement of the load block in the north direction. It can be seen that the difference between $\Delta_{DT3}$ and $\Delta_{DT3,x}$ is well over 10% for much of the duration of the test; and thus, adjustments need to be applied to the measurements from DT3. In comparison, the adjustments needed for the vertical displacement measurements from DT4 and DT5 remain small throughout the test with the largest difference being less than 2.0%. Figure 6.230 shows the percent difference between the measured and adjusted displacements for DT4 and DT5 plotted against the beam chord rotation, $\theta_b$. It can be seen that away from the origin, the maximum differences between the unadjusted and adjusted measurements from DT4 and DT5 remain less than 0.5%.

The measurements from DT3 require larger adjustments than those from DT4 and DT5 since the changes in the string angle for DT3, which occur due to the applied vertical displacements of the load block, are much larger than the changes in the string angles for DT4 and DT5, which occur due to the gap opening displacements at the beam ends. In evaluating the results from Test 4, adjusted measurements are used for $\Delta_{DT3}$; however, the measurements for $\Delta_{DT4}$ and $\Delta_{DT5}$ are not adjusted. Figure 6.231 plots $\Delta_{DT4}$, $\Delta_{DT5}$, and $\Delta_{DT3,x}$ against the beam chord rotation, $\theta_b$. It can be seen in Figures 6.231(a) and 6.231(b) that the vertical displacement of the load block at the north and south ends are nearly the same.

Combining these displacements, the $x$-displacement, $y$-displacement, and rotation of the load block centroid can be determined as described in Chapter 5 and shown in Figures 6.232 and 6.233. Figure 6.232(a) plots the $y$-displacement versus the $x$-displacement showing the path of the load block centroid during the test. As the
subassembly is displaced under positive (i.e., clockwise) and negative (i.e., counterclockwise) rotations, the load block is pushed north (in the $x$-direction, away from the reaction block) due to gap opening at the beam ends. After each cycle, the load block returns to its initial position with minimal residual displacements. Note that unlike Tests 1 – 3, Figures 6.232(a) and 6.2331(a) show that the $x$-direction displacement of the load block centroid is symmetric during the positive and negative rotations of the subassembly. The load block displaces symmetrically in the $y$-direction as well as shown in Figures 6.232(a), 6.232(d), and 6.233(b). Finally, the rotation of the load block is shown to remain small throughout the duration of the test as shown in Figures 6.232(b) and 6.233(c). The load block centroid rotation remains below 0.003 radians indicating that the two hydraulic actuators moved near simultaneously.
Figure 6.227: Test 4 load block horizontal displacements – (a) measured, $\Delta DT_3$; (b) adjusted, $\Delta DT_3,x$; (c) percent difference.
Figure 6.228: Test 4 load block north end vertical displacements – (a) measured, $\Delta DT4$; (b) adjusted, $\Delta DT4, y$; (c) percent difference.
Figure 6.229: Test 4 load block south end vertical displacements –
(a) measured, $\Delta DT5$; (b) adjusted, $\Delta DT5,y$; (c) percent difference.

Figure 6.230: Test 4 percent difference between measured and adjusted displacements –
(a) DT4; (b) DT5.
Figure 6.231: Test 4 load block displacements versus beam chord rotation – (a) $\Delta_{\text{DT4}}-\theta_b$; (b) $\Delta_{\text{DT5}}-\theta_b$; (c) $\Delta_{\text{DT3,x}}-\theta_b$. 
Figure 6.232: Test 4 load block centroid displacements – (a) $x$-$y$ displacements; (b) rotation; (c) $x$-displacements; (d) $y$-displacements.
Figure 6.233: Test 4 load block centroid displacements versus beam chord rotation – (a) $x$-displacement-$\theta_b$; (b) $y$-displacement-$\theta_b$; (c) rotation-$\theta_b$.

6.5.11 Reaction Block Displacements and Rotations

String pots DT6 – DT8 are used to measure the vertical $y$-displacements and the horizontal $x$-displacement of the reaction block. Figure 6.234 shows the measurements from DT6 – DT8 for the duration of the test. Since the reaction block is tied to the strong floor, the measured displacements remain very small throughout the test. Due to these small displacements and the use of lead cables for each string pot, the change in angle that the string undergoes during testing is very small. Thus, it can be assumed that no
adjustments are needed for the displacements measured from the reaction block displacement transducers. Figure 6.235 plots the measurements from DT6 – DT8 against the beam chord rotation.

The $x$-displacement, $y$-displacement, and rotation of the reaction block centroid can be determined (see Chapter 5) as shown in Figure 6.236 and plotted against the beam chord rotation in Figure 6.237. It is concluded that the vertical displacements of the reaction block do not have a significant effect on the displacements of the test structure (e.g., the beam chord rotation), and the test results are presented with the reaction block displacements taken as zero (i.e., the measured displacements of the reaction block are ignored in investigating the response of the subassembly). Note that the horizontal displacements of the reaction block are significant when determining the total elongation of the post-tensioning tendon; and thus, are included in those calculations.
Figure 6.234: Test 4 reaction block displacements – (a) $\Delta_{DT6}$; (b) $\Delta_{DT7}$; (c) $\Delta_{DT8}$.
Figure 6.235: Test 4 reaction block displacements versus beam chord rotation – (a) $\Delta DT7 - \theta_b$; (b) $\Delta DT8 - \theta_b$; (c) $\Delta DT6 - \theta_b$. 
Figure 6.236: Test 4 reaction block centroid displacements – (a) $x$-$y$ displacements; (b) rotation; (c) $x$-displacements; (d) $y$-displacements.
6.5.12 Contact Depth and Gap Opening at Beam-to-Wall Interfaces

The beam contact depth and gap opening displacements are measured at the beam-to-reaction-block interface using displacement transducers DT11 – DT13. As described in Chapter 5, these LVDTs rotate with the beam; and thus, their measurements may need to be adjusted to determine the gap opening displacements in the horizontal $x$-direction.
Figures 6.238 through 6.240 plot the measured displacements $\Delta_{DT11}$, $\Delta_{DT12}$, and $\Delta_{DT13}$, respectively, the corresponding adjusted x-displacements $\Delta_{DT11,x}$, $\Delta_{DT12,x}$, and $\Delta_{DT13,x}$, respectively, and the percent differences between the measured and adjusted displacements. The results indicate that the adjusted measurements are less than 0.04% different from the original measurements; and thus, $\Delta_{DT11}$, $\Delta_{DT12}$, and $\Delta_{DT13}$ can be taken as the displacements in the x-direction. Figure 6.241 plots the measured data from DT11, DT12, and DT13 against the beam chord rotation.

The maximum average concrete compressive strain in the beam-to-wall contact regions can be calculated by dividing the measured displacements from DT11 and DT13 with the gauge length (i.e., the distance from the LVDT ferrule insert in the beam to the reaction plate ferrule insert in the wall test region; see Chapter 5). For Test 4, the maximum average compressive strain is 0.0029. Note that this measurement includes the compressive strain occurring in the fiber-reinforced grout at the beam-to-wall interface as well as the patched concrete deformations in the wall test region of the reaction block.
Figure 6.238: Test 4 beam-to-reaction-block interface top LVDT displacements – (a) measured, $\Delta_{DTIL}$; (b) adjusted, $\Delta_{DTIL,x}$; (c) percent difference.
Figure 6.239: Test 3 beam-to-reaction-block interface middle LVDT displacements – (a) measured, $\Delta_{DT12}$; (b) adjusted, $\Delta_{DT12,x}$; (c) percent difference.
Figure 6.240: Test 4 beam-to-reaction-block interface bottom LVDT displacements – (a) measured, $\Delta DT13$; (b) adjusted, $\Delta DT13,x$; (c) percent difference.
Using the measured data, the contact depth and the largest (i.e., at the beam top and bottom) gap opening displacements at the beam-to-reaction-block interface can be determined following the procedures in Chapter 5. Figures 6.242(a) and 6.243(a) show the results based on the measured data from DT11 – DT13 (method 1); Figures 6.242(b) and Figure 6.243(b) show the results based on the measured data from RT2, DT11, and DT13 (method 2); Figures 6.242(c) and 6.243(c) show the results based on the measured data from RT2 and DT12 (method 3); Figures 6.242(d) and 6.243(d) show the results based on the beam chord rotation, $\theta_b$ and the measurements from DT12 (method 4); and
Figures 6.242(e) and 6.243(e) show the results based on the beam chord rotation, $\theta_b$, and the measurements from DT11 and DT13 (method 5). Note that in Chapter 5, methods 2 and 3 use the data from RT1; however, because of the drift in this measurement for Test 4, the data from RT2 is used. Each ○ marker in Figures 6.242 and 6.243 indicates the contact depth or gap opening displacement at the peak of a loading cycle up to a beam chord rotation of 3.33%.

Looking at Figure 6.242, it can be stated that the contact depth results obtained using the five methods are somewhat different but show similar trends. There is a rapid reduction in the contact depth up to a beam chord rotation of about 2.0%. After this rotation, the contact depth remains relatively stable due to the nonlinear behavior of the concrete in compression and varies between $5.0 – 2.0$ in. ($127 – 51$ mm) [about $30 – 10\%$ of the beam depth] under positive rotations and varies between $4.0 – 1.0$ in. ($102 – 25$ mm) [about $20 – 5.0\%$ of the beam depth] under negative rotations. Note that the difference in contact depths under positive and negative rotations may be due to the patched region in the wall test region. In comparison, the gap opening results using the five methods in Figure 6.243 are reasonably similar and the increase in gap opening with the rotation of the beam is very close to linear.

Figure 6.244 shows a continuous plot of the largest gap opening displacements determined from method 4 (using the beam chord rotation and DT12) against the beam chord rotation. Similarly, Figure 6.245(a) plots the beam contact depth from method 4 against the beam chord rotation as continuous data. Furthermore, Figures 6.245(b) and 6.245(c) plot the contact depth for the 2.25% and 3.33% beam chord rotation cycles, respectively. It can be seen that unlike Tests 1 and 2, the contact depth behavior during
the 2.25% cycle in Test 4 is similar to the behavior during the 5.0% cycle as was also seen in Test 3. It is unclear if the instruments and/or beam shifted to skew the data or if this is the actual behavior of the beam. Furthermore, the gap opening and gap contact depth plots in Figures 6.244 and 6.245 could have been affected by the use of $\theta_b$ (i.e., chord rotation) instead of $\theta_{RTI}$ (i.e., local rotation) to determine the behavior at the south end of the beam. The use of the beam midheight transducer DT12 instead of the extreme beam top or bottom transducer (DT11 or DT13) may also have affected the results, especially the contact depth plots, in Figure 6.245 during small beam rotations, which are very sensitive to the measurements. Thus, the estimated contact depths at small rotations ($\theta_b < 0.25\%$) should be used with caution.

In addition to the LVDT measurements, the gap opening at the south end of the beam were taken using a ruler. These ruler measurements are listed in Table 6.4 and shown using + markers in Figure 6.243. It can be seen that the LVDT gap opening measurements in Figure 6.243 are reasonably close to the ruler measurements in Table 6.4.
Figure 6.242: Test 4 contact depth at beam-to-reaction-block interface – (a) method 1 using $\Delta DT11$, $\Delta DT12$, and $\Delta DT13$; (b) method 2 using RT2, DT11, and DT13; (c) method 3 using RT2 and DT12; (d) method 4 using $\theta_b$ and DT12; (e) method 5 using $\theta_b$, DT11, and DT13.
Figure 6.243: Test 4 gap opening at beam-to-reaction-block interface – (a) method 1 using $\Delta DT_{11}$, $\Delta DT_{12}$, and $\Delta DT_{13}$; (b) method 2 using RT2, DT11, and DT13; (c) method 3 using RT2 and DT12; (d) method 4 using $\Delta DT_{12}$ and $\theta_b$; (e) method 5 using $\theta_b$, DT11, and DT13.
Figure 6.244: Test 4 gap opening at beam-to-reaction-block interface using method 4.

Figure 6.245: Test 4 contact depth at beam-to-reaction-block interface using method 4 – (a) entire data; (b) 2.25% beam chord rotation cycle; (c) 3.33% beam chord rotation cycle.
TABLE 6.4

RULER MEASUREMENTS OF GAP OPENING AT SOUTH BEAM END

<table>
<thead>
<tr>
<th>Nominal Rotation (%)</th>
<th>Gap Opening, $\Delta g$ [in. (mm)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0175</td>
<td>0.005 (0.13)</td>
</tr>
<tr>
<td>0.25</td>
<td>0.013 (0.33)</td>
</tr>
<tr>
<td>0.35</td>
<td>0.025 (0.635)</td>
</tr>
<tr>
<td>0.50</td>
<td>0.06 (1.52)</td>
</tr>
<tr>
<td>0.75</td>
<td>0.125 (3.18)</td>
</tr>
<tr>
<td>1.0</td>
<td>0.1875 (4.76)</td>
</tr>
<tr>
<td>1.5</td>
<td>0.25 (6.35)</td>
</tr>
<tr>
<td>2.25</td>
<td>0.375 (9.53)</td>
</tr>
<tr>
<td>3.33</td>
<td>0.5 (12.7)</td>
</tr>
</tbody>
</table>

6.5.13 Wall Test Region Local Concrete Deformations

The reaction block confined concrete deformations near the beam-to-wall interface of the wall test region are measured using displacement transducers DT14 and DT15. Figure 6.246 plots the time history results from the top (DT14) and bottom (DT15) transducers, and Figure 6.247 plots the measured data from DT14 and DT15 against the load block beam chord rotation. As expected, the concrete deformations are mostly compressive (negative) due to the compression stresses that are transferred through the contact region from post-tensioning and gap opening.

From Figure 6.246, the maximum average concrete compressive strain in the wall test region can be calculated by dividing the measured deformations with the gauge length (i.e., the distance from the LVDT ferrule insert in the wall test region to the...
reaction plate; see Chapter 5). For Test 4, the maximum average concrete compressive strain is 0.006, which is greater than the expected unconfined (cover) concrete crushing strain of 0.004. Both of these strain values might have been affected by to the grout-patch in the wall test region.

![Reaction Block LVDT Displacement](image)

Figure 6.246: Test 3 wall test region concrete deformations – (a) DT14; (b) DT15.

![Reaction Block LVDT Displacement](image)

Figure 6.247: Test 4 wall test region concrete deformations versus beam chord rotation – (a) $\Delta_D T 14 - \theta_b$; (b) $\Delta_D T 15 - \theta_b$. 
6.5.14 Beam Looping Reinforcement Longitudinal Leg Strains

Figures 6.248 and 6.249 show the strain gauge measurements for the top and bottom horizontal legs of the east and west No. 6 mild steel looping reinforcing bars in the beam. The locations of these strain gauges can be found in Chapter 5. The initial beam post-tensioning force results in a small compressive strain in the bars at the beginning of the test. Upon lateral loading of the subassembly, the largest tensile strains occur, as expected, in the gauges closest to the angle-to-beam connection bolts [i.e., gauges 6(1)T-E, 6(1)T-W, 6(1)B-E, and 6(1)B-W]. The measurements in the gauges away from the angle-to-beam connection decrease with distance from this critical location.

To provide a better understanding of the strain measurements in the horizontal legs of the beam looping reinforcement, the Δ and □ markers in Figures 6.248(a) and 6.248(b) for gauges 6(1)T-E and 6(1)T-W correspond to positive and negative chord rotation peaks for the beam, respectively, and the ○ markers indicate zero rotation positions. To provide further insight into the results, Figures 6.250 and 6.251 show the strains plotted against the beam chord rotation. In the positive (i.e., clockwise) rotation direction, the strains in the top bars increase in tension as the gap opens at the top south corner of the beam and the top angle is pulled in tension. In the negative (i.e., counterclockwise) direction, the strains in the bottom bars increase in tension and the top bars go into compression due to the closing of the gap.

The maximum strains in the four gauges closest the critical section (i.e., the angle-to-beam connection) remain well below the yield strain of the longitudinal steel (ε_{iy} =
0.00283) from the material tests in Chapter 4; and thus, it is concluded that the amount of mild steel reinforcement used to transfer the angle forces into the beam is adequate.

Figure 6.248: Test 4 beam looping reinforcement top longitudinal leg strains –
(a) $\varepsilon_{6(1)T-E}$; (b) $\varepsilon_{6(1)T-W}$; (c) $\varepsilon_{6(2)T-E}$; (d) $\varepsilon_{6(2)T-W}$; (e) $\varepsilon_{6(3)T-E}$; (f) $\varepsilon_{6(3)T-W}$; (g) $\varepsilon_{6MT-E}$; (h) $\varepsilon_{6MT-W}$. 
Figure 6.249: Test 4 beam looping reinforcement bottom longitudinal leg strains –
(a) $\epsilon_{6(1)B-E}$; (b) $\epsilon_{6(1)B-W}$; (c) $\epsilon_{6(2)B-E}$; (d) $\epsilon_{6(2)B-W}$; (e) $\epsilon_{6(3)B-E}$; (f) $\epsilon_{6(3)B-W}$; (g) $\epsilon_{6MB-E}$; (h) $\epsilon_{6MB-W}$.
Figure 6.249 continued.

Figure 6.250: Test 4 beam looping reinforcement top longitudinal leg strains versus beam chord rotation – (a) $\varepsilon_{6(1)T-E} - \theta_b$; (b) $\varepsilon_{6(1)T-W} - \theta_b$; (c) $\varepsilon_{6(2)T-E} - \theta_b$; (d) $\varepsilon_{6(2)T-W} - \theta_b$; (e) $\varepsilon_{6(3)T-E} - \theta_b$; (f) $\varepsilon_{6(3)T-W} - \theta_b$; (g) $\varepsilon_{6MT-E} - \theta_b$; (h) $\varepsilon_{6MT-W} - \theta_b$. 
Figure 6.250 continued.
Figure 6.251: Test 4 beam looping reinforcement bottom longitudinal leg strains versus beam chord rotation – (a) $\varepsilon_{6(1)B-E}$-$(\theta_b);$ (b) $\varepsilon_{6(1)B-W}$-$(\theta_b);$ (c) $\varepsilon_{6(2)B-E}$-$(\theta_b);$ (d) $\varepsilon_{6(2)B-W}$-$(\theta_b);$ (e) $\varepsilon_{6(3)B-E}$-$(\theta_b);$ (f) $\varepsilon_{6(3)B-W}$-$(\theta_b);$ (g) $\varepsilon_{6MB-E}$-$(\theta_b);$ (h) $\varepsilon_{6MB-W}$-$(\theta_b).$
6.5.15 Beam Transverse Reinforcement Strains

Figure 6.252 shows the strain measurements from gauges 6SE(I)-E, 6SE(E)-E, 6SE(I)-W, and 6SE(E)-W placed on the transverse (i.e., vertical) legs of the east and west No. 6 looping reinforcing bars at the south end of the beam. The locations of these strain gauges can be found in Chapter 5. To give more insight into the measurements, Figure 6.253 plots the strain data against the load block beam chord rotation. It can be seen that the strain gauge readings remain well below the yield strain of the reinforcing steel ($\varepsilon_{\text{y}} = 0.00283$, see Chapter 4) throughout the experiment, demonstrating that the design of the transverse reinforcement at the beam ends is adequate. Note that the angle-to-beam connection bolts may also have acted as transverse reinforcement in the beam; however, this could not be confirmed from the test results since the connection bolts were not instrumented.

Similarly, Figure 6.254 shows the strain measurements from gauges MH-E and MH-W placed on the vertical legs of the No. 3 transverse hoop at the beam midspan. The
locations of these gauges can be found in Chapter 5. To give more insight into the measurements, Figure 6.255 plots the strain data against the beam chord rotation. As expected, the results indicate that the maximum strains in the midspan hoop remained well below the yield strain of the steel ($\varepsilon_{\text{hy}} = 0.00240$, see Chapter 4); and thus, the use of nominal transverse reinforcement within the span of the beam is adequate.

Figure 6.252: Test 4 beam looping reinforcement vertical leg strains – (a) $\varepsilon_{6SE(E)-E}$; (b) $\varepsilon_{6SE(E)-W}$; (c) $\varepsilon_{6SE(I)-E}$; (d) $\varepsilon_{6SE(I)-W}$.
strain gauge 6SE (E)-E removed from Beam #4

strain gauge 6SE (E)-W removed from Beam #4

Figure 6.253: Test 4 beam looping reinforcement vertical leg strains versus beam chord rotation – (a) $\varepsilon_{6SE(E)-E}$; (b) $\varepsilon_{6SE(E)-W}$; (c) $\varepsilon_{6SE(I)-E}$; (d) $\varepsilon_{6SE(I)-W}$.

strain gauge MH-E removed from Beam #4

Figure 6.254: Test 4 beam midspan transverse hoop reinforcement strains – (a) $\varepsilon_{MH-E}$; (b) $\varepsilon_{MH-W}$.
6.5.16 Beam Confined Concrete Strains

Figures 6.256 and 6.257 show the measurements from the strain gauges placed on the No. 3 support bars inside the hoop confined concrete at the south end of the beam. The locations of these strain gauges can be found in Chapter 5. For further insight into the strain gauge readings, Figures 6.258 and 6.259 plot the strain data against the beam chord rotation. The results show that there is a small compressive strain at the beginning of the test due to the initial post-tensioning force. As the beam is rotated in the positive (i.e., clockwise) direction, the compression strains in the bottom bars increase due to the transfer of the contact stresses through the bottom corner of the beam. In the opposite (i.e., counterclockwise) direction, the strains in the top and bottom bars reverse due to the reversal of the load. It can be seen that the strain gauge measurements remain below the yield strain $\varepsilon_{ry} = 0.00240$ of the No. 3 support bars in tension and below the expected crushing strain $\varepsilon_{cu} = 0.004$ of the unconfined concrete crushing strain in compression.
strain gauge 3THT-(1) removed from Beam #4

strain gauge 3THB-(1) removed from Beam #4

strain gauge 3THT-(2) removed from Beam #4

strain gauge 3THB-(2) removed from Beam #4

(a) \( \varepsilon_{3THT-(1)} \); (b) \( \varepsilon_{3THB-(1)} \); (c) \( \varepsilon_{3THT-(2)} \); (d) \( \varepsilon_{3THB-(2)} \).
Figure 6.257: Test 4 No. 3 bottom hoop support bar strains –
(a) $\varepsilon_{3BHB-(1)}$; (b) $\varepsilon_{3BHT-(1)}$; (c) $\varepsilon_{3BHB-(2)}$; (d) $\varepsilon_{3BHT-(2)}$. 

strain gauge 3BHT-(1) removed from Beam #4.
strain gauge 3THT-(1) removed from Beam #4

strain gauge 3THB-(1) removed from Beam #4

strain gauge 3THT-(2) removed from Beam #4

strain gauge 3THB-(2) removed from Beam #4

Figure 6.258: Test 4 No. 3 top hoop support bar strains versus beam chord rotation –
(a) $\varepsilon_{3THT-(1)} - \theta_b$; (b) $\varepsilon_{3THB-(1)} - \theta_b$; (c) $\varepsilon_{3THT-(2)} - \theta_b$; (d) $\varepsilon_{3THB-(2)} - \theta_b$. 
Figure 6.259: Test 4 No. 3 bottom hoop support bar strains versus beam chord rotation – (a) $\varepsilon_{3BHB-(1)} - \theta_b$; (b) $\varepsilon_{3BHT-(1)} - \theta_b$; (c) $\varepsilon_{3BHB-(2)} - \theta_b$; (d) $\varepsilon_{3BHT-(2)} - \theta_b$.

### 6.5.17 Beam End Confinement Hoop Strains

Figures 6.260 and 6.261 show the measurements from the strain gauges placed on the vertical legs of the bottom layer No. 3 confinement hoops at the south end of the beam. The locations of these strain gauges can be found in Chapter 5. For further insight into the measurements, Figure 6.262 and 6.263 plot the strain data against the beam chord rotation. The angle-to-beam connection bolt forces result in a small compressive strain in
the vertical hoop steel at the beginning of the test. Throughout the test, the measured confinement hoop strains remain small.

Figure 6.260: Test 4 beam end confinement hoop east leg strains – (a) $\varepsilon_{1HB-E}$; (b) $\varepsilon_{2HB-E}$; (c) $\varepsilon_{3HB-E}$; (d) $\varepsilon_{4HB-E}$. 
Figure 6.261: Test 4 beam end confinement hoop west leg strains – (a) $\varepsilon_{1HB-W}$; (b) $\varepsilon_{2HB-W}$; (c) $\varepsilon_{3HB-W}$; (d) $\varepsilon_{4HB-W}$. 
Figure 6.262: Test 4 beam end confinement hoop east leg strains versus beam chord rotation – (a) $\varepsilon_{1HB-E}-\theta_b$; (b) $\varepsilon_{2HB-E}-\theta_b$; (c) $\varepsilon_{3HB-E}-\theta_b$; (d) $\varepsilon_{4HB-E}-\theta_b$. 

The figures show plots of strain versus beam chord rotation for different strain gauges removed from Beam #4.
6.5.18 Wall Test Region Confined Concrete Strains

As described previously, the strain gauge wires coming out of the reaction block were all severed during the removal of the steel casting mold. Thus, no measurements were recorded for the wall test wall region confined concrete strains.
6.5.19 Wall Test Region Confinement Hoop Strains

Similar to above, no measurements were recorded for the wall test region confinement hoop strains since the strain gauge wires were severed.

6.5.20 Crack Patterns

The crack propagation from testing is marked on the beam during each cycle of displacement (e.g., see Figures 6.202, 6.203, and 6.204). In addition, the crack propagation is recorded by approximately copying the crack patterns manually onto paper. Figure 6.264 shows the hand-drawn crack patterns recorded for Test 4 at the end of the last cycle from selected displacement increments (see Appendix F for crack sheets).

Figure 6.264: Test 4 crack patterns – (a) $\theta_b = 0.125\%$; (b) $\theta_b = 0.175\%$; (c) $\theta_b = 0.25\%$; (d) $\theta_b = 0.35\%$; (e) $\theta_b = 0.5\%$; (f) $\theta_b = 0.75\%$; (g) $\theta_b = 1.0\%$; (h) $\theta_b = 1.5\%$; (i) $\theta_b = 2.25\%$; (j) $\theta_b = 3.33\%$. 
Figure 6.264 continued.