Experimental Evaluation of a Multi-Story Post-Tensioned Coupled Shear Wall Structure

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ABSTRACT

This paper discusses the design and experimental evaluation of a novel seismic-resistant reinforced concrete (RC) coupled shear wall system. In this system, the widely-used unbonded post-tensioned floor slab construction method is adapted to couple (i.e., link) two RC wall piers, providing significant performance and construction benefits over conventional RC coupling beams in high seismic regions. Previous experiments of post-tensioned coupled wall structures are limited to floor-level coupling beam subassemblies. The current paper extends the available research to multi-story structures by presenting the design of an 8 story prototype test specimen consisting of two C-shaped shear walls. The design is validated through the testing of a simplified 15% scale specimen in the laboratory. The experimental specimen includes the foundation, the first three floors of the shear walls, and the associated coupling beams. The upper stories of the building are simulated with hydraulic jacks that supply the appropriate bending moment, shear, and axial forces at the top of the laboratory structure. This paper compares the measured load displacement response of the laboratory structure with predictions from design models. Experimental and design predictions of several key behavior parameters are shown to match well. Future work involves the construction and testing of large scale (40%) specimens to validate the approach. Ultimately, the measured information from the test specimens will be used in the development of validated design procedures and modeling/prediction tools for multi-story post-tensioned coupled wall structures.

INTRODUCTION

Reinforced concrete coupled shear walls are a commonly used lateral load system in high-rise condominium, hotel, and office towers in the U.S. These structures are constructed by placing coupling (i.e., link) beams at the floor and roof levels to transfer forces between the wall piers and also dissipate energy during an earthquake. The resulting system is stronger than the sum of the wall piers, allowing for
efficiency in design; however, the detailing and construction of the coupling beams pose significant challenges due to the presence of large reversed cyclic rotation demands under large shear forces.

To achieve ductile behavior, the most common practice for RC coupling beams in seismic regions is the diagonally reinforced system (Barney et al. 1978; Tassios et al. 1996; Bristowe 2000; Galano and Vignoli 2000; Canbolat et al. 2005). As shown in Fig. 1(a), the placement of two intersecting groups of diagonal reinforcing bars through the beam and into the wall piers is a major challenge in practice. In comparison, the overall vision of the current project is a new type of coupled wall structure that utilizes post-tensioned coupling beams [Fig. 1(b)]. The new system eliminates the diagonal reinforcement by using a combination of high-strength unbonded post-tensioning (PT) steel with top and bottom horizontal mild steel (U.S. Grade 60) reinforcing bars to develop the coupling forces. The PT force is provided by multi-strand tendons placed inside ungrouted ducts (to prevent bond between the steel and concrete) through the center of each coupling beam and the wall piers. The mild steel bars at the beam ends are designed to yield and dissipate energy while the PT tendon gives the system self-centering capability, thus creating an efficient structure.

The current research involves investigators from the University of Notre Dame, UT Tyler, and the Lehigh University NEES site. The current paper describes the testing of a 15% scale multi-story structure at UT Tyler. Previous experiments on the use of unbonded post-tensioning to couple RC walls are limited to isolated floor level subassembly studies (Weldon and Kurama 2007, 2009, 2010). The UT Tyler test is the first known test of a multi-story post-tensioned coupled wall system.

**PROTOTYPE BUILDING DESIGN AND DETAILS**

To form a basis for the experimental investigation of the 40% and 15%-scale PT coupled shear wall structures, a full-scale 8-story prototype building was designed for a site in Los Angeles, California with a calculated seismic response coefficient of $C_s=0.136g$. The plan and elevation views of this structure are shown in Figures 2(a) and 2(b). The primary lateral load resistance is provided by the coupled core wall at
the center of the building – two C-shaped shear walls connected by coupling beams at each floor level. This core includes two openings to simulate the location of elevator shafts and stairwells in a typical office building. These openings were centered inside the core in the north-south direction to help eliminate any asymmetric behavior under loading. The configuration, dimensions, and detailing of the prototype building were chosen with the assistance of Magnusson Klemencic Associates (MKA).

Figure 2. Prototype structure: (a) building plan; (b) building elevation; (c) wall base details; (d) coupling beam details

Figure 2(c) shows the wall pier reinforcement details at the base of the structure. The reinforcement plan was selected so that it was similar to the typical reinforcement used in a conventional coupled shear wall design. The post-tensioned coupling beam details are given in Figure 2(d). The post-tensioning force is provided by 16 - 12.7 mm (0.5 in.) diameter PT strands, placed inside two ungrouted ducts to prevent bonding to the concrete. Energy dissipation in the structure is provided by 3 – No. 19 (U.S. No. 6) bars at the top and bottom of the beams. These bars are wrapped in plastic at the beam ends to prevent bond between the steel and concrete. The length of
this wrapped section is selected to limit the maximum strains in the steel during reversed-cyclic loading.

To determine the design forces for the prototype structure, the Equivalent Lateral Force (ELF) procedure from ASCE 7 was used with an assigned R-factor of 6.0. The structure had a calculated period of $T = 0.74$ seconds. The ELF procedure resulted in a design total base moment and total base shear force, which were then distributed to the individual components of the coupled core wall structure by making a number of design selections. First, a coupling degree of 30% was chosen, meaning that 30% of the design base moment is to be carried by the coupling action between the two wall piers. This coupling moment is distributed to the coupling beams as a design shear force and corresponding moment at the beam ends, with the remaining base moment distributed evenly between the two wall piers. The reinforcement details of the wall pier base and PT coupling beams were then selected to satisfy these design forces.

**15% SCALE SPECIMEN DESCRIPTION**

Several key decisions were made to create the 15% scale model of the prototype building within the capabilities of the laboratory. First, scaling the C-shaped wall piers directly would result in walls in the experimental specimen only 2.7 inches thick. Instead, the section modulus of the scaled C-shape was matched using a wall that was rectangular in shape, as shown in Figure 3 and Table 1. Rather than scaling the actual reinforcement from the prototype structure, the base moment of the prototype structure was scaled, and then the flexural steel of the laboratory walls was designed using basic reinforced concrete principles, resulting in the selection of the No. 19 (U.S. No. 6) and No. 22 (No. 7) bars shown in Figure 3, and the moment capacities shown in Table 1. Target base moments were derived from a DRAIN-2DX analytical model described in the next section. Similarly, the base shear forces of the prototype structure wall piers were scaled, and design using basic RC principles resulted in the No. 13 (No. 4) hoops in the wall piers shown in Figure 3(c).

The prototype structure has two coupling beams per story – in the 15% structure these beams were combined, thus matching the scaled cross-sectional area and resulting in constructible beam geometry. Energy dissipation steel was not included in the beams. The moment and shear capacity design of the coupling beams followed a similar approach to the pier design – demands from the prototype structure were scaled, and basic prestressed and reinforced concrete design principles were used to select the steel for the 15% structure – see Figure 3 and Table 1.

Figure 4 shows the 15% laboratory structure. Loading was accomplished using a lateral jack that supplied the story shears for all three stories constructed plus the resultant shear for the upper five stories. In an actual structure, lateral load from an earthquake would be applied to both piers independently – in the UT Tyler structure this load was lumped and applied through one jack only. The gravity load from the
upper stories was applied using tensioned cables within ducts in the piers. Displacements were measured with a string potentiometer attached to the left pier at the height of the application of the lateral load.

Additional axial force (both tensile and compressive) in the piers caused by coupling was provided by jacks attached to cables at the top of the wall piers that allowed both tension and compression to be applied. These cables were connected to steel beams attached to the tops of the walls. By attaching these cables at a distance from the centerline of the wall piers, the associated jacks were also used to provide the overturning moment at the top of each pier caused by the upper stories. Forces in all three hydraulic jacks were measured using calibrated load cells.

The foundation, wall piers, and coupling beams were all cast separately. The flexural reinforcement in the wall piers was cast with a 17.8 cm (7 in.) extension protruding from the base, and the base moment connection for each pier was created by grouting these bars into anchors embedded in the foundation.
Table 1 – UT Tyler Scaling Details

<table>
<thead>
<tr>
<th></th>
<th>Section Modulus (m³)</th>
<th>Moment of Inertia (m⁴)</th>
<th>Base Moment (kft)</th>
<th>Base Shear (kip)</th>
<th>Area of Story Coupling Beams (m²)</th>
<th>Moment of Story Coupling Beams (k-in)</th>
<th>Shear of Story Coupling Beams (kip)</th>
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<tbody>
<tr>
<td>Prototype Structure (T) Pier (C) Pier</td>
<td>183231</td>
<td>19092676</td>
<td>46812</td>
<td>1424</td>
<td>864</td>
<td>9442</td>
<td>196.6</td>
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<tr>
<td>Target (15% Scale) (T) Pier (C) Pier</td>
<td>618.4</td>
<td>9666</td>
<td>158</td>
<td>158</td>
<td>17.3</td>
<td>19.44</td>
<td>31.9</td>
</tr>
<tr>
<td>UT Tyler Structure (T) Pier (C) Pier</td>
<td>1152</td>
<td>13824</td>
<td>164.4</td>
<td>34.2</td>
<td>34.2</td>
<td>20</td>
<td>37.2</td>
</tr>
</tbody>
</table>

The applied loads (lateral, overturning moment, and axial) in the experiment were scaled directly from those of the prototype building, as can be seen in later figures.

RESULTS

Figure 5 shows the lateral load versus displacement history for the experimental test. One full load reversal was applied to the structure until failure in both directions. To achieve the full load reversal, the lateral jack and the two vertical jacks were moved from the positions shown in Figure 4 after failure in the first loading direction. Locations where unloading is evident in the plot occurred when the displacement capacity of the lateral loading jack was exceeded, so the jack was depressurized while spacers were added to increase the total structure lateral travel. The hysteresis of the structure exhibited a smaller amount of re-centering capability than was expected, which was due to the relatively small PT forces that were achieved in the coupling beam strands (described later).

The analytical results in Figure 5 are as follows. The “target” line was created through analysis of the 15%-scaled prototype structure in DRAIN-2DX. The DRAIN-2DX model consists of a series of nodes connected by nonlinear beam-column elements that are discretized into fibers with areas and material properties that
accurately depict the concrete and steel present in the structure. The model used the ELF procedure to determine the lateral force distribution to be applied for a displacement-controlled pushover analysis of the full 8-story structure. The “ABAQUS” line was created through finite element analysis in ABAQUS of the UT Tyler structure as constructed. The philosophy behind this model was to simulate a simplified design office approach to analysis of the post-tensioned coupled wall system. The geometry matched Figures 3 and 4, including application of gravity loads through post-tensioning cables. When tensioning the cables in the experimental specimen, the relatively short length of the cables and poor construction control meant that the design initial gravity force in the gravity cables and post-tensioning force in the beam cables was not met – see Table 2. This resulted in an as-built structure that had approximately 75% of the intended gravity load based on the tributary areas of the prototype structure. The ABAQUS model reflects the as-built forces. The “DRAIN” line was created through analysis of the actual UT Tyler geometry and as-built initial cable forces with modifications discussed later. In contrast with the target model, this model used a force-controlled analysis with the prescribed loading history that was applied during the Tyler experiment in the three jacks. To simulate the overturning moment, a rigid element was created that extends from the top of each wall pier to the location of the vertical jacks where the loads are applied.

![Graph](image)

Figure 5. Lateral Load vs. Lateral Drift Behavior of 15% Structure

The initial stiffness of the test structure matches predictions. The tested structure did not achieve the strength predicted in the target and ABAQUS lines. It is believed that the bars grouted into the anchors in the foundation pulled out at a relatively low load, although these couplers were Type II seismically rated. Detailed inspection of this joint is on-going as the structure is currently being disassembled. Figure 6 shows ABAQUS results varying the assumed effectiveness of the moment steel crossing the base joint. The initial slopes and strengths of these curves match portions of the measured results, indicating that this is the most likely explanation for the low
strength of the test structure. The “DRAIN” results in all of the figures herein reflect a structure whose base moment steel is assumed 67% effective, and this model matches the experimental results closely.

Table 2 – Beam PT Strand Forces – Design vs. As-Built

<table>
<thead>
<tr>
<th></th>
<th>Target (lb)</th>
<th>As-Built (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension Pier Gravity</td>
<td>56600</td>
<td>37700</td>
</tr>
<tr>
<td>Compression Pier Gravity</td>
<td>56600</td>
<td>44600</td>
</tr>
<tr>
<td>Beam Cable - 1st Story</td>
<td>17000</td>
<td>454</td>
</tr>
<tr>
<td>Beam Cable - 2nd Story</td>
<td>17000</td>
<td>3284</td>
</tr>
<tr>
<td>Beam Cable - 3rd Story</td>
<td>17000</td>
<td>8338</td>
</tr>
</tbody>
</table>

Figure 6. ABAQUS Parametric Investigation

Figure 7(a) shows the applied overturning moment at the top of each pier versus the lateral drift and shows that the applied loads in the experiment match those from the analysis models well. Overturning moment in the figure is calculated at the top of the experimental specimen (above the third story). Unloading and load reversal portions of the measured data have been removed from the plot for clarity. The major difference between the target structure and the others occurs due to the method of vertical load application. Because the overturning moment and axial force (from coupling effects above the third story) are applied through one cable at a distance from each pier centerline, the ratio of the axial force to overturning moment is constant (the distance from the centerline of each pier to the cable associated with each pier) throughout the test. In the target model the correct (varying) ratio between axial force and overturning moment was used. The impact of the constant ratio seems small in the other models and experimental results.

Figure 7(b) shows the applied total axial force in each pier calculated by algebraically adding the gravity cable forces to the vertical jack forces. The force is reported at the top of the experimental specimen (above the third story). The largest deviation from target behavior occurs in the tension pier. This may be related to the simplification of the wall piers from C-shaped to rectangular – the stiffness properties of the two piers
in the UT Tyler test are more similar to each other than those in the target structure. The ABAQUS model varies from the target, experimental and DRAIN behavior because in this model the force in the gravity cables becomes large at large drifts. This may be related to the assumed concrete material behavior in the ABAQUS model, where elastic-plastic properties were assumed to be symmetric in the tension and compression regimes for simplicity. The DRAIN model concrete material properties are more realistically non-symmetric, with a compressive strength of 64 MPa (9275 psi) (derived from cylinder testing of the experimental concrete), and a conservatively estimated tensile strength of 1.4 MPa (200 psi).

Figure 7. UT Tyler Analytical Study: (a) overturning moment vs. lateral drift; (b) wall pier axial forces vs. drift

Figure 8 shows the forces in the beam PT cables. The experiment and models show that the greatest force develops in the third floor cable. The trend of increasing PT force in each beam with increasing drift is captured in all of the approaches. As noted, the target structure differs from the other plots because the initial post-tensioning forces in the UT Tyler beams were smaller than expected (see Table 2). The explanation for the ABAQUS (which has the as-built initial forces) difference from the experiment with increasing drift is similar to that for the total pier axial force—the symmetric compression-tension concrete material behavior is likely the culprit. The DRAIN model shows excellent agreement with the experiment. Thus, the experiment is a successful first validation of the design and modeling approaches for post-tensioned coupled shear walls developed in this research program.

FUTURE WORK

Construction of the first of two 40%-scale physical laboratory specimens is currently underway at Lehigh University. These laboratory specimens will include the first three floors, tributary slabs, and foundations of the prototype coupled core wall, representing the most critical regions of the structures. The other (less critical) regions of the structures will be simulated in the computer, resulting in a hybrid physical-computational research platform. The forces and displacements from the computer model will be applied to the physical structure using a total of 7 actuators.
and 4 gravity jacks, simulating the behavior of the upper 5 stories of the 8-story building. Figure 9(a) shows a three-dimensional rendering of the 40%-scale experimental setup in the laboratory, while Figure 9(b) illustrates the construction progress on the first specimen.

Figure 8. Beam Post-Tensioning Forces: (a) 1st story; (b) 2nd story; (c) 3rd story

The 40% scale specimen will be monitored at multiple locations with two- and three-dimensional digital image correlation (DIC) on selected faces of the walls, beams, and floors to gather full-field surface deformation data during the tests (McGinnis et al. 2011). In the 15% scale tests described herein, 4 locations were monitored with DIC – the bases of both piers, the side faces of each of the coupling beams, and the overall structure. These results will be presented in a future paper. Ultimately, the results of the 15% and 40% studies are expected to lead to the development of validated design procedures and modeling/prediction tools for the new system.

SUMMARY AND CONCLUSIONS

The experiment described is the first known instance of testing of a multi-story coupled wall structure rather than the testing of individual components. Testing of
the 15% scale specimen had three major goals (1) develop and test design procedures for post-tensioned coupled shear walls; (2) validate modeling approaches for these structures; and (3) develop and validate approaches to application of multiple DIC sensors to the same structural test. Comments on goal (3) will be forthcoming in a future paper that discusses the results of the multiple DIC systems deployed as part of this test (whose results could not be included herein due to space limitations).

The results presented herein show that the design procedure for the post-tensioned coupled wall system yielded a structure that performed as expected, and that the analytical models yielded predictions in good agreement with measured behavior. There are several limitations of the current experiment with the three major being: (1) the scaling of the specimen meant that C-shaped walls became rectangular shaped, and energy dissipation steel in the coupling beams could not be included; (2) the forces in the beam post-tensioning cables were less in the current experiment than as designed, and (3) the tension reinforcement necessary to resist the base moment in each of the piers pulled out of the structural couplers at lower than the design load. Nevertheless, matching between DRAIN-2DX predictions of the structural behavior across several key parameters such as lateral load versus drift and beam post-tensioning forces was good. Prediction based on a simpler ABAQUS model also matched the measured experimental data reasonably well. Notwithstanding the limitations noted above, the structure was scaled appropriately.

The structure exhibited behavior consistent with some of the potential benefits of a post-tensioned coupled wall system, with some self centering capability, and damage limited mostly to the toes of the coupling beams. Obviously with no energy dissipating steel, the total energy dissipated during the fully reversed lateral load cycle was not as great as would be expected in an actual structure.
Although the data from all of the multiple DIC sensors used to monitor this test could not be shown herein due to space limitations (it will be presented in a future paper), the approach used appears to have been successful.

ACKNOWLEDGEMENTS

This research is funded by the National Science Foundation (NSF) under Grant No. CMMI 10-41598. The support of Dr. Joy Pauschke, NSF Program Director, is gratefully acknowledged. The authors thank David C. Fields – Senior Associate, Amy D. Haaland – Engineer, and Joshua Mouras – Engineer, of the Magnusson Klemencic Associates, Inc. for their help on the design of the full-scale prototype structure. The authors also thank Kenneth Bondy for his contributions to the project. The donation of materials from Suncoast Post-Tension is gratefully acknowledged. Construction of the 15% scale test specimen could not have been completed without the efforts of Michael Lisk, an undergraduate student at UT Tyler. The opinions, findings, and conclusions expressed in this paper are those of the authors and do not necessarily reflect the views of the NSF or the individuals acknowledged above.

REFERENCES


