Behavior of Precast Concrete Shear Walls for Seismic Regions: Comparison of Hybrid and Emulative Specimens

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Abstract: This paper discusses the lateral load behavior of two, 0.40-scale, hybrid, precast concrete shear wall test specimens and the behavior of a third precast specimen designed to emulate monolithic cast-in-place RC shear walls. The walls had identical overall geometry and were constructed by placing rectangular precast panels across horizontal joints. The hybrid walls used mild steel bars [Grade 400 (U.S. Grade 60)] and high-strength unbonded posttensioning (PT) strands for lateral resistance, whereas the emulative wall used only mild steel bars. The mild steel bars crossing the base joint were designed to yield and provide energy dissipation, with the PT steel in the hybrid walls reducing the residual displacements of the structure. The mild steel bars at the base of the emulative wall and one of the hybrid walls used Type II mechanical splices, while the other hybrid wall used continuous bars grouted into the foundation. Because of the lack of PT steel, the emulative wall developed a large residual uplift at the base joint, resulting in excessive horizontal slip and strength degradation. The behavior of the hybrid wall with Type II splices was also limited, which occurred because of the pullout of the mild steel bars. In contrast, the hybrid wall with continuous mild steel bars showed superior restoring, energy dissipation, and ductile behavior over larger lateral displacements. The results show the potential for the use of precast walls in seismic regions, while also revealing important detailing considerations. DOI: 10.1061/(ASCE)ST.1943-541X.0000755.

CE Database subject headings: Concrete; Post tensioning; Precast concrete; Reinforced concrete; Bars; Tests; Seismic effects; Shear walls; Comparative studies.

Author keywords: Post-tensioned concrete; Precast concrete; Reinforced concrete; Spliced bars; Seismic tests; Emulative walls; Hybrid walls; Shear walls.

Introduction

This paper discusses the behavior of three precast concrete shear wall specimens under combined lateral and gravity loading. Two walls investigate a hybrid system that uses longitudinal (i.e., vertical) mild steel bars [Grade 400 (U.S. Grade 60)] and high-strength unbonded posttensioning (PT) strands for lateral resistance across horizontal joints. The third specimen is an emulative wall with only mild steel reinforcement (i.e., no PT steel) to imitate the behavior of monolithic cast-in-place RC shear walls. One of the walls is shown in Fig. 1(a), with the general features for the hybrid and emulative systems shown in Figs. 1(b and c), respectively.

Under lateral loads to the nonlinear range, the desired primary displacement mode in both types of precast walls occurs through a gap opening at the base-panel-to-foundation joint. In comparison, the flexural and shear deformations of the wall panels are not significant. The mild steel bars crossing the base joint, referred to as the energy-dissipating (ED) steel, are designed to yield in tension and compression, and provide energy dissipation through the gap opening/closing behavior of the wall under reversed-cyclic lateral loading. A predetermined length of these bars is unbonded at the base joint (by wrapping the bars with plastic sleeves) to limit the steel strains and prevent low-cycle fatigue fracture. The ED bars of the emulative wall and one of the hybrid walls used Type II mechanical splice connections (as defined in Chapter 21 of ACI 318-11 [American Concrete Institute (ACI) 2011]) with matching bars cast inside the foundation, whereas the ED steel of the other hybrid wall featured continuous bars grouted into the foundation.

The PT steel in the hybrid system is provided by multistrand tendons inside ungrouted ducts to prevent bond with the concrete. Thus, the tendons are connected to the wall only at end anchorages. Upon unloading, the PT steel (in addition to the gravity loads on the wall) provides a restoring force to close the gap at the base joint, thus reducing the residual (i.e., permanent) displacements of the structure. The use of unbonded tendons reduces the strand strains as well as the tensile stresses transferred to the concrete as the tendons elongate under lateral loading, thus, preventing or delaying the yielding of the strands and reducing the cracking of the wall panels.

Objectives and Scope

Significant limitations exist on the use of precast walls in seismic regions of the United States. Chapter 21 of ACI 318-11 specifies that “a reinforced concrete structural system not satisfying the requirements of this chapter shall be permitted if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this chapter.” Hybrid walls fall into this category of...
The minimum experimental evidence required for the U.S. code validation of precast concrete walls as special RC shear walls is specified in ACI ITG-5.1-07 (ACI 2007). Design guidelines and requirements for walls satisfying ACI ITG-5.1-07 can be found in ACI ITG-5.2-09 (ACI 2009). Previous research on seismic precast structures goes back to the PREcast Seismic Structural Systems (PRESSS) Research Program in the early 1990s (Priestley 1991). This program included both frame and wall systems; however, an overview of the work on frame structures is outside the scope of this paper. Previous work on precast shear walls includes emulative walls (Holden et al. 2003), fully posttensioned walls with little energy dissipation (Kurama et al. 2002; Perez et al. 2007; Restrepo 2003), and posttensioned walls with supplemental energy dissipation to reduce the seismic displacements (Pristley et al. 1999; Perez et al. 2004; Kurama 2000, 2001; Ajраб et al. 2004; Marriott et al. 2008; Hamid and Mander 2010).

To date, limited experimental results are available for hybrid precast walls with internally placed mild steel and PT steel reinforcement for lateral resistance (Restrepo 2003; Holden et al. 2003). More recently, Smith et al. (2011) tested a multipanel hybrid wall (referred to as Specimen HW1 in this paper). Different from the hybrid walls described in Restrepo (2003) and Holden et al. (2003), Specimen HW1 was a relatively low-rise structure and included an upper panel-to-panel joint in addition to the base joint. Furthermore, Specimen HW1 was specifically designed and tested to satisfy the validation requirements in ACI ITG-5.1-07, while requiring little deviation from conventional U.S. precast concrete construction. Additional studies on building structures with hybrid precast shear walls are described in Schoettler et al. (2009) and Nagae et al. (2011); however, full results from these tests have not yet been published.

This paper reports on hybrid wall detailing improvements based on the performance of Specimen HW1, which suffered from premature crushing of the confined concrete at the wall toes. In addition, comparisons of the hybrid walls with an emulative precast wall are provided. Ultimately, these results provide experimental, analytical, and design evidence on the behavior of hybrid precast concrete shear walls with practical construction details via a study conducted according to ACI ITG-5.1-07 and ACI 318-11, while also noting important design considerations for the wall reinforcement at the base.

**Test Setup and Specimen Properties**

The general design parameters and geometry were kept consistent among the hybrid walls (referred to as Specimens HW2 and HW3) and the emulative wall (referred to as Specimen EW) described in this paper, and the hybrid wall (Specimen HW1) from Smith et al. (2011). Each specimen was designed as a special RC shear wall [within an ASCE/SEI 7-05 “building frame system” (ASCE 2005)] for a 4-story prototype parking garage with an approximate footprint of 3,770 m² (40,600 ft²) in Los Angeles, California. The walls were designed based on ASCE/SEI 7-05, ACI 318-11, and ACI ITG-5.2-09 for a full-scale base moment of 27,120 kN·m (20,000 kip-ft). The design was conducted at two levels of drift: (1) the design-level drift, \( \Delta_{\text{d,dr}} \), determined according to ASCE/SEI 7-05 and (2) the maximum-level drift, \( \Delta_{\text{um}} \), which was taken equal to the validation-level drift, \( \Delta_{\text{v,dr}} = 2.30\% \), prescribed by ACI ITG-5.1-07. More information on the hybrid wall design procedure and the results from a pretest analytical study can be found in Smith et al. (2011, 2012). The design of the emulative wall followed a similar procedure but using no PT steel. All three specimens were designed with minimal detailing and overstrength to test the limits of the design procedures (e.g., no capacity reduction factor was used in the axial-flexural design of the base joint and the reinforcement areas were selected close to the required areas).

**Overall Geometry and Test Configuration**

Each specimen was tested at 0.40-scale and featured two panels: the base panel representing the first story of the prototype structure and
the upper panel representing the second through fourth stories. It was possible to model the upper story panels of the 4-story prototype wall as a single panel because the joints between these panels were designed to have no gap opening or other type of nonlinear behavior. At the horizontal joints, an approximately 13 mm (0.5 in.) thick, fiber-reinforced, dry-pack grout pad (with polypropylene microfilament fibers at 0.065% by volume per manufacturer’s recommendations) was used for panel alignment and construction tolerance purposes. The 0.40-scale wall length, \( l_w \), was 243 cm (96 in.); the base panel height, \( h_{pb} \), was 145 cm (57.5 in.); the wall thickness, \( t_w \), was 15.9 cm (6.25 in.); and the wall height-to-length aspect ratio was 2.25.

The lateral load was applied 3.66 m (12 ft) from the wall base (near the resultant location of the first mode inertial forces), resulting in a relatively low wall base moment-to-shear ratio of \( M_b/V_b \approx 1/5l_w \). A reversed-cyclic lateral displacement history was used, with three cycles at each increment. In addition to the self-weight of the wall [38 kN (8.5 kips)], an external axial load of approximately 325 kN (73 kips) was applied at the top of each specimen (resulting in a total axial load ratio of \( N_w/f_cA_g = 0.023 \)). Two response-monitoring systems were used (Smith et al. 2011): (1) a conventional system with 23 displacement sensors, 5 rotation sensors, 9 load cells, and 34 strain gauges; and (2) a three-dimensional digital image correlation (3D-DIC) system using a noncontact optical technique. The 3D-DIC system measured the in-plane and out-of-plane displacements of the base panel over a region of one-half the panel length and one-half the panel height from the south toe. A portion of the foundation also was included in the 3D-DIC field of view, providing unprecedented information on the response across the base joint.

**Reinforcement Details**

The base panel details for Specimen HW2 are shown in Fig. 2 (courtesy of the Consulting Engineers Group, Inc.), with a listing of the important features of all three walls provided in Table 1. The reinforcement in Specimen HW2 was identical to that in Specimen HW1, except for the following:

1. The first confinement hoop was placed closer to the bottom of the base panel [1.9 cm instead of 5.0 cm from the bottom (4.75 cm instead of 12.5 cm full scale)]. This change was made because of the premature crushing of the confined concrete at the toes of Specimen HW1.
2. The welded wire fabric used as distributed panel reinforcement in Specimen HW1 was replaced with distributed No. 10 (U.S. No. 3) deformed bars. This change was made to allow better

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**Table 1. Specimen Parameters**

<table>
<thead>
<tr>
<th>Specimen number</th>
<th>Number of strands and diameter</th>
<th>( f_{pt}/f_{pu} )</th>
<th>Eccentricity ( \epsilon_p ) (cm)</th>
<th>Size number</th>
<th>Eccentricity ( \epsilon_e ) (cm)</th>
<th>Wrapped length (cm)</th>
<th>( f_{en}/f_{e} )</th>
<th>Continuity at base</th>
<th>( l_h ) (cm)</th>
<th>( s_{hoop} ) (cm)</th>
<th>( s_{bot} ) (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HW1</td>
<td>3 – 1.27 cm</td>
<td>0.54</td>
<td>( \pm 23 )</td>
<td>19</td>
<td>( \pm 7.5, 15 )</td>
<td>25</td>
<td>0.64</td>
<td>Spliced</td>
<td>40</td>
<td>8.3</td>
<td>5.0</td>
</tr>
<tr>
<td>HW2</td>
<td>3 – 1.27 cm</td>
<td>0.54</td>
<td>( \pm 23 )</td>
<td>19</td>
<td>( \pm 7.5, 15 )</td>
<td>25</td>
<td>0.64</td>
<td>Spliced</td>
<td>40</td>
<td>8.3</td>
<td>1.9</td>
</tr>
<tr>
<td>HW3</td>
<td>3 – 1.27 cm</td>
<td>0.54</td>
<td>( \pm 28 )</td>
<td>19</td>
<td>( \pm 8.9, 19 )</td>
<td>38</td>
<td>0.48</td>
<td>Continuous</td>
<td>40</td>
<td>7.6</td>
<td>1.9</td>
</tr>
<tr>
<td>EW</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>22</td>
<td>( \pm 79, 91, 104 )</td>
<td>56</td>
<td>0.73</td>
<td>Spliced</td>
<td>20</td>
<td>8.3</td>
<td>1.9</td>
</tr>
</tbody>
</table>

Note: \( f_{pt} \) = average initial PT strand stress; \( f_{pu} \) = design ultimate strength of strand (1.862 MPa); \( \epsilon_p \) = distance of individual PT tendon or ED bar, respectively, from wall centerline; \( \epsilon_{en} \) = maximum expected (design) ED bar strain at \( \Delta_{en} = 2.3\% \); \( \epsilon_e \) = strain at maximum (peak) strength of ED steel from monotonic material testing; \( l_h \) = confinement region length at wall toes (measured from center-to-center of bar); \( s_{hoop} \) = confinement hoop spacing (measured from center-to-center of bar); \( s_{bot} \) = first hoop distance from bottom of base panel (measured to center of bar).

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**Fig. 2.** Base panel details (elevation view) for Specimen HW2
control of concrete cover, which was small due to the scaled dimensions.

Similarly, the following changes were made from Specimen HW2 to Specimen HW3:

1. The confinement hoop spacing was reduced from 8.3 cm (3.25 in.) to 7.6 cm (3.0 in.). This change was made to increase the effectiveness of the concrete confinement at the wall toes.

2. The ED bar continuity details were changed from Type II mechanical splices (permitted in ACI ITG-5.2-09 and Chapter 21 of ACI 318-11) to continuous bars grouted into the foundation. This change was made because of the pullout of the ED bars from the splices in Specimen HW2.

3. The ED bars were placed at ±19 and ±8.9 cm (±7.5 and ±3.5 in.) instead of ±15 and ±7.5 cm (±6.0 and ±3.0 in.) from the wall centerline. This was necessary for the placement of the continuously grouted bars through the oversize ducts inside the foundation. As a consequence, the locations of the PT tendons also were changed [±28 cm (±11 in.) instead of ±23 cm (±9.0 in.) from the wall centerline].

4. The wrapped length of the ED bars was increased from 25 cm (10 in.) to 38 cm (15 in.). This change was made to reduce the maximum expected bar strains, \( \varepsilon_{um} \) (Table 1).

The reinforcement for the base joint in Specimen EW consisted of six No. 22 (U.S. No. 7) bars utilizing Type II splices, with three bars at each end of the wall [±104, ±91, and ±79 cm (±41, ±36, and ±31 in.) from the centerline]. The bars were wrapped over 56 cm (22 in.) at the bottom of the base panel. The distributed steel in the wall panels consisted of No. 10 bars. Across the upper panel-to-panel joint, six No. 22 bars were used, with three bars at each end of the wall [±109, ±97, and ±84 cm (±43, ±38, and ±33 in.) from the centerline]. As in the hybrid walls, these bars were designed not to yield so as to limit any gap opening or slip at the upper joint. A 7.6 cm (3.0 in.) length of the bars was wrapped at the bottom of the upper panel to prevent strain concentrations in the steel. Because the PT steel was eliminated resulting in reduced compression forces, the concrete confinement at the wall toes extended over a smaller length \( l_b = 20 \text{ cm (8.0 in.)} \) instead of \( l_b = 40 \text{ cm (16.0 in.)} \) in the hybrid walls. The confinement cages for the specimens used closed hoops to represent typical U.S. practice. The use of alternate confinement details [e.g., external armor plates (Holden et al. 2003; Hamid and Mander 2010)] was not investigated by this research project as recommended by a relevant industry advisory panel.

### Material Properties

Fig. 3(a) shows the measured stress-strain behavior of the ASTM A416 PT strand [design ultimate strength, \( f_{pu} = 1,862 \text{ MPa (270 ksi)} \)]. The strand was tested using sand-grip anchors as described in Walsh and Kurama (2010). The yield stress of 1,620 MPa (235 ksi) was determined from the proportionality limit in the stress-strain relationship. Similarly, Figs. 3(b and c) show the behaviors of the ASTM A706 Grade 400 ED steel and the ASTM A615 Grade 400 confinement hoop steel, respectively. The PT and ED steel strains were measured using an MTS Model 634.2SE-24 extensometer with 5.1-cm (2.0-in.) gauge length. The extensometer was removed prior to fracture, with the remaining strains calculated from the relative displacements of the testing machine crossheads.

Selected measured steel properties are listed in Table 2. Note that the ED bars in Specimen EW did not have a distinct yield plateau; therefore, the yield strength of these bars was taken as the

### Table 2. Material Properties

<table>
<thead>
<tr>
<th>Specimen number</th>
<th>PT strand</th>
<th>ED steel</th>
<th>Confinement hoop steel</th>
<th>Base panel concrete</th>
<th>Base joint grout</th>
<th>ED bar grout</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( f_{pu} ) (MPa)</td>
<td>( \varepsilon_{pu} ) (%)</td>
<td>( f_{y} ) (MPa)</td>
<td>( \varepsilon_{y} ) (%)</td>
<td>( f_{pu} ) (MPa)</td>
<td>( \varepsilon_{pu} ) (%)</td>
</tr>
<tr>
<td>HW1</td>
<td>1620</td>
<td>0.83</td>
<td>448</td>
<td>0.21</td>
<td>608</td>
<td>13.7</td>
</tr>
<tr>
<td>HW2</td>
<td>1620</td>
<td>0.83</td>
<td>448</td>
<td>0.21</td>
<td>608</td>
<td>13.7</td>
</tr>
<tr>
<td>HW3</td>
<td>1620</td>
<td>0.83</td>
<td>462</td>
<td>0.23</td>
<td>647</td>
<td>12.4</td>
</tr>
<tr>
<td>EW</td>
<td>1620</td>
<td>0.83</td>
<td>419</td>
<td>0.35</td>
<td>741</td>
<td>10.8</td>
</tr>
</tbody>
</table>

Note: Results are averaged from three material samples each. \( f_{pu}, f_{y}, f_{pu} = \) yield strength; \( \varepsilon_{pu}, \varepsilon_{y}, \varepsilon_{pu} = \) strain at \( f_{pu}, f_{y}, \) [circles in Fig. 3(a–c)]; \( f_{pu}, \varepsilon_{pu} = \) ultimate (maximum) strength; \( \varepsilon_{y}, \varepsilon_{pu} = \) strain at \( f_{pu}, f_{y} [\text{triangles in Figs. 3(b and c)}]; \varepsilon_{pu}, \varepsilon_{pu} = \) ultimate strain at 0.85\( f_{pu} \) [inverted triangles in Figs. 3(b and c)]; \( f_{pu} = \) maximum (peak) strength of unconfined concrete or grout at 28 days; \( f_{pu} = \) maximum (peak) strength of unconfined concrete or grout at day of wall testing; \( E_{s,ad} = \) secant stiffness of unconfined concrete or grout at day of wall testing.

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The wall panels were cast flat on the precast production bed. The design compressive strength of the unconfined concrete was $f'_c = 41$ MPa (6.0 ksi) and the design confined-concrete strength was approximately 62 MPa (9.0 ksi). The measured strengths of the unconfined base panel concrete [from $10 \times 20$ cm (4.0 \times 8.0 in.) cylinders], the fiber-reinforced joint grout [from $5.1 \times 5.1$ cm (2.0 \times 2.0 in.) cubes], and the grout for the ED bar ducts and splices (from $5.1 \times 5.1$ cm cubes) are listed in Table 2. The secant stiffnesses for the concrete and dry-pack joint grout in Specimen HW3 were measured from $7.6 \times 15$ cm (3.0 \times 6.0 in.) cylinders using an Epsilon 3542RA extensometer (Epsilon Technology Corporation, Jackson, WY) with 5.1-cm (2.0-in.) gauge length. The stiffness was determined from two points on the stress-strain relationship (at a strain of 0.00005 cm/cm and the strain corresponding to 0.40 times the measured peak stress). The 28-day dry-pack joint grout strength was specified to be within $\pm 20\%$ of the 28-day unconfined concrete strength of the base panel to provide a matching bearing bed for the panel concrete at the base joint. Note that the concrete in Specimens HW2, HW3, and EW was considerably stronger than that in Specimen HW1, which did not reach the design strength [weaker concrete contributed to the premature failure of Specimen HW1; Smith et al. (2011)].

**Analytical Modeling**

Two different analytical models were employed for the hybrid walls (Smith et al. 2011): (1) a fiber-element model using DRAIN-2DX (Prakash et al. 1993), and (2) a finite-element model using ABAQUS (ABAQUS 2009). As described in Smith et al. (2011, 2012), the shear deformations of the wall panels were small despite the low $M_p/V_p$ ratio. In the fiber model, these deformations were represented using an effective shear area of $A_{sh} = 0.8A_{gross}$, which remained constant during the analysis (where $A_{gross} =$ panel gross cross-section area). The emulative Specimen EW was modeled [Fig. 4(a)] following similar techniques as the hybrid walls; however, a zero-length connection element was added to model the horizontal slip that occurred at the base joint (discussed in the section “Slip at Horizontal Joints”). As shown in Fig. 4(b), an idealized bilinear shear force-slip relationship was used in the connection element based on the measured shear force-slip behavior of the wall.

**Analytical and Measured Behaviors of Wall Specimens**

Figs. 5(a–c) show the measured base shear force, $V_p$, versus wall drift, $\Delta_w$, behaviors for Specimens HW2, HW3, and EW, respectively. The wall drift, $\Delta_w$ (positive with the wall displaced southward), was measured as the relative lateral displacement of the wall between the lateral load location and the top of the foundation divided by the height to the lateral load. Specimen HW3 was able to sustain two cycles at $\Delta_w = \pm 2.30\%$ (i.e., the validation-level drift) followed by a greater cycle of $\Delta_w = \pm 2.95\%$. The wall behaved in a reasonably symmetrical manner in the positive and negative directions, and exhibited excellent recentering and considerable energy dissipation. While crushing of the confined concrete was observed at the wall toes, the total strength loss from the overall peak base shear force during the test to the peak force during the final cycle at $\Delta_w = \pm 2.95\%$ was 19.9\% and 13.8\% in the positive and negative directions, respectively, which are within the 20\% strength loss limit prescribed by ACI ITG-5.1-07.

Specimen HW2 was subjected to three cycles at the $\pm 2.30\%$ validation-level drift; however, the strength and energy dissipation of the wall at this drift did not satisfy ACI ITG-5.1-07. The wall failed during the $\Delta_w = \pm 1.55\%$ cycles, when the ED bars pulled out from the ACI 318-11 Type II mechanical splice connectors inside the foundation because of failure of the grout within the connectors. These connectors provided ease of erection for the base panel; however, pullout of the ED bars caused the steel strains to be smaller than designed, limiting the lateral strength and energy dissipation of the wall. Using the measured PT steel stresses, applied gravity load, and neutral axis length at the last cycle of $\Delta_w = \pm 2.30\%$, the lateral strength of the wall ignoring the ED steel was estimated to range from 358 to 371 kN (81 to 84 kips), indicated by the shaded region in Fig. 5(a). Comparing this range with the measured strength of 385 kN (87 kips), it was confirmed that Specimen HW2 was essentially behaving as a fully posttensioned wall by the end of the test. Note that the grout used in the ED bar splices satisfied the splice manufacturer’s specifications and the splice itself satisfied the requirements in ACI 318-11 and AC133 [International Code Council (ICC) Evaluation Service 2010] for Type II connectors. The reason for the splice failure was that the ED bars in the wall specimens were subjected to greater strains and over a significantly larger number of cycles than required to classify a Type II connection according to ACI 318-11 and AC133 (Smith et al. 2012).

![Fig. 4. Fiber-element (DRAIN-2DX) modeling of emulative Specimen EW: (a) model elevation; (b) zero-length slip element at wall base](image-url)
The emulative Specimen EW, which was tested to provide a baseline comparison for the hybrid walls, failed after only three cycles to $\Delta_w = \pm 1.15\%$. Until the tensile yielding of the ED bars, the emulative wall behaved similarly to the hybrid walls (except for more widespread hairline cracking in the emulative system). However, failure of the emulative specimen occurred relatively early, because of uplift of the wall from the foundation. Upon unloading from tensile yielding of the ED bars, the restoring capability of the wall was limited. Without the PT force, the restoring effect of the 325 kN (73 kips) gravity load applied at the top of the wall and the 38 kN (8.5 kips) self-weight of the wall was not sufficient to yield the bars back in compression and close the gap at the base. This resulted in a residual gap along the entire joint when the wall returned to $\Delta_w = 0\%$. Upon reloading, the nonlinear behavior and failure of the wall was dominated by excessive in-plane horizontal shear-slip at the base, with a relatively small gap opening compared to the hybrid walls. The behavior of the wall in the positive and negative drift directions was somewhat unsymmetrical, due to different amounts of slip in the two directions.

The predicted $V_f-\Delta_w$ behaviors of Specimens HW2, HW3, and EW using the fiber-element model are depicted in Figs. 5(d–f), respectively. It can be seen that the measured behavior of Specimen HW3 is captured quite well; however, there are significant discrepancies for Specimen HW2 because the pullout of the ED bars was not modeled. The analytical model of Specimen EW was subjected to a monotonic lateral pushover analysis rather than the cyclic displacement history used during the experiment. This cyclic loading history could not be replicated in the analysis because of the residual gap (uplift) that formed along the base joint, resulting in numerical convergence problems. It can be seen that the analytical results provide a reasonable match to the envelope of the measured $V_f-\Delta_w$ behavior of the structure.

Progression of Damage in Specimen HW2

Fig. 6(a) shows Specimen HW2 at the third cycle to $\Delta_w = +2.30\%$ (note the gap at the north end of the base joint). Similar to Specimen HW1 (Smith et al. 2011), the damage to Specimen HW2 was limited to the base panel, with no concrete cracking or crushing in the upper panel and no significant gap opening or shear-slip at the upper joint. Cracking in the base panel initiated during the third cycle to $\Delta_w = -0.27\%$ (in Specimen HW1, cracking initiated during the first cycle to $\Delta_w = +0.40\%$) and remained small during the test [typically less than 1 mm (0.04 in.) wide].

Cover concrete spalling in Specimen HW2 began during the $\Delta_w = 0.80\%$ cycles (compared to $\Delta_w = 0.40\%$ in Specimen HW1). This improvement was related to the use of No. 10 bars instead of welded wire fabric for the panel distributed steel, which allowed for more consistent cover thickness in Specimen HW2 (because of the scaled thickness of the wall). Crushing of the confined concrete did not occur in Specimen HW2 (compared to Specimen HW1, where confined concrete crushing began during the $\Delta_w = 1.15\%$ cycles). The better performance of the confined concrete in Specimen HW2 as compared to Specimen HW1 was related to the following: (1) closer placement of the first hoop to the bottom of the base panel, (2) smaller compression at the wall toes due to the pullout (smaller tension) in the ED bars, and (3) higher concrete strength.

Progression of Damage in Specimen HW3

Fig. 6(b) shows Specimen HW3 at $\Delta_w = +2.95\%$. Similar to Specimens HW1 and HW2, the damage was limited to the base panel. Cracking in the base panel began during the first cycle to $\Delta_w = +0.55\%$ (the design-level drift), which is later than in Specimens HW1 and HW2. Cover concrete spalling in Specimen HW3 initiated during the $\Delta_w = \pm 0.80\%$ cycles. Significant crushing of the confined concrete at the wall toes was not present until $\Delta_w = +2.30\%$, after which strength degradation was evident in the $V_f-\Delta_w$ behavior [Fig. 5(b)]. During the larger drift cycles, the terminated ends of the distributed horizontal No. 10 (U.S. No. 3) panel reinforcement started to delaminate from the wall at the base, accelerating the progression of concrete spalling along the wall length. Because of the relatively small thickness of the 0.40-scale specimen, these bars had a design clear cover of only 1.3 cm (0.5 in.) and were terminated outside the confinement cages on either face of the panel [not satisfying Section 21.9.6.4(e) of ACI 318-11]. Upon
spalling of the cover concrete, the terminated ends of the horizontal bars at the wall toes became exposed, causing delamination. Increased clear cover or terminating the horizontal bars inside the confinement cages would likely result in a reduction of cover spalling over the wall length.

Near the completion of the test, outward bowing of the longer, horizontal legs of the confinement hoops was observed (the hoops had a relatively large length-to-width ratio of $l_{\text{hoop}}/w_{\text{hoop}} = 3.56$). While a crosstie was present within the confinement detailing, this tie did not directly engage the hoop. Rather, as permitted by ACI 318-11, each crosstie and hoop separately engaged the vertical bars within the confinement cage, making the tie ineffective to prevent the longitudinal legs of the hoop from bowing out. Using two overlapping hoops with smaller length-to-width ratios would likely result in better performance of the confined concrete.

**Progression of Damage in Specimen EW**

Fig. 6(c) shows Specimen EW at the third cycle to $\Delta_w = +1.15\%$. While the crack widths in the wall panels generally remained small, the cracking in Specimen EW was considerably more extensive than in the hybrid specimens and extended high into the upper panel. Cracking of the wall panels began during the first cycle to $\Delta_w = +0.13\%$, which is well before the design-level drift of $\Delta_w = 0.60\%$ (note that the walls had slightly different design-level drifts due to differences in the measured concrete properties) and prior to the cracking of the hybrid specimens. Cover concrete spalling did not begin until the first cycle to $\Delta_w = +1.15\%$, but significant deterioration to the concrete at the wall toes progressed rapidly (and earlier than in the hybrid specimens), resulting in large strength and stiffness degradation. Under load reversal with increasing slip, the concrete around the ED bars deteriorated because of the shear force transfer from the bars to the surrounding concrete, eventually resulting in localized splitting of the base panel around the bars. Similar to the hybrid specimens, no significant slip or gap opening was observed in the upper panel-to-panel joint.

**ED Bar Behavior**

As described previously, the ED bars had plastic-wrapped unbonded lengths to limit the tensile steel strains and, thus, prevent low-cycle fatigue fracture, while also allowing significant yielding of the bars as the walls were displaced. Fig. 7(a) shows the measured (using strain gauges placed within the wrapped unbonded length) and predicted (using the fiber-element model) steel strains for the north intermediate ED bar in Specimen HW2. The differences in the steel strains for the positive and negative drift directions are due to the different elongations of the bar as the wall was displaced in each direction. For clarity, the strains are shown at the peak of the third cycle of each wall drift increment instead of the full cyclic history. Due to gauge failure, measurements could only be taken up to a maximum strain of 0.015 cm/cm at $\Delta_w = +1.15\%$. As designed, the bars yielded relatively early in the loading history. The predicted strains significantly overestimated the measured strains because the pullout of the bars was not captured in the analysis.

Similarly, Fig. 7(b) shows the measured and predicted strains for the north intermediate ED bar in Specimen HW3. Measurements could be taken only up to a maximum strain of 0.0275 cm/cm at $\Delta_w = +1.55\%$ prior to gauge failure. As in Specimen HW2, the bars yielded relatively early in the loading history; however, testing of Specimen HW3 was completed with no undesirable behavior of the steel (i.e., no bar fracture, observable slip, or buckling). Because no bar pullout occurred in this specimen, the measured and predicted strains compare favorably.

The measured and predicted strains for the north intermediate bar in Specimen EW are depicted in Fig. 7(c). Because the reinforcement in this wall was placed at the ends of the panel rather than near the centerline, larger bar elongations were expected. Therefore, a longer 56 cm (22 in.) wrapped length was used to limit the maximum bar strains. Again, the bars were designed to yield relatively early; however, because the horizontal shear-slip at the base joint contributed significantly to the total wall drift (as described subsequently), the gap opening displacements at the base were smaller than the hybrid specimens; and, thus, the ED bar strains also remained relatively small. At the maximum drift of $\Delta_w = +1.15\%$, the measured strain was 0.0135 cm/cm, which is significantly smaller than in Specimen HW3. The fiber-element model, which included the shear-slip displacements at the base, provided a good match to the measured strains.

To quantify the energy dissipation of the structure, ACI ITG-5.1-07 uses the energy dissipation ratio, $\beta$, which is defined as “the ratio of the measured energy dissipated by the test module during reversing cyclic displacements between given measured drift angles to
the maximum theoretical energy that can be dissipated for the same drift angles.” ACI ITG-5.1-07 requires that $\beta$ be not less than 0.125 at the validation-level drift. The solid and dashed lines in Fig. 7(d) show the measured and predicted $\beta$ ratios, respectively, of the three specimens at the last cycle for each drift level. Because of the ED bar pullout, the energy dissipation of Specimen HW2 rapidly decreased after the cycles to $D_w = 56\%$ and did not satisfy the ACI ITG-5.1-07 minimum $\beta$ at the end of the test. Because the bar pullout was not included in the analytical model, the predicted results significantly overestimate the measured results. In comparison, Specimens HW3 and EW satisfied the ACI ITG-5.1-07 minimum $\beta$ at moderate drift levels and continued to exceed the requirement until the end of the test. As expected, Specimen EW demonstrated considerably larger energy dissipation than Specimen HW3 because approximately 40% of the base moment of the hybrid wall was provided by the PT steel, which remained mostly linear-elastic during the test. While the energy dissipation of Specimen EW could not be determined from the monotonic analysis, the predicted and measured $\beta$ ratios for Specimen HW3 show good agreement.

**PT Steel Behavior**

Figs. 8(a and b) show the measured and predicted PT forces for Specimen HW2 by plotting the normalized average stresses in the north and south PT tendons (calculated as the sum of the measured strand forces divided by $A_{\text{p}}f_{\text{pu}}$, where $A_{\text{p}}$ = total area of the strands in each tendon and $f_{\text{pu}} = 1,862\$ MPa). Similar results for Specimen HW3 are shown in Figs. 8(c and d). Consistent with the design expectations, the PT tendons remained essentially linear-elastic until $\Delta_w = 1.55\%$, which was possible because the strands were unbonded over their length. Losses in the PT stresses, which occurred primarily because of a small amount of nonlinear behavior in the strand-anchorage system, can be seen during the large displacement cycles and are clearly visible upon unloading from the final drift cycle. Despite these losses, the residual drift of each wall at the end of the test was negligible [Figs. 5(a and b)]. The analytical models matched the measured PT steel behavior reasonably well; however, the PT stress losses were not fully captured by the analyses.

**Gap Opening and Neutral Axis Length**

Consistent with the design expectations, each specimen opened a significant gap at the base joint while the gap opening at the upper joint was negligible. Figs. 9(a and b) show the measured and predicted, respectively, maximum gap opening displacements at the base joint (i.e., vertical size of gap, upward positive) at the extreme north and south ends of Specimens HW2 and HW3. Similar results for Specimen EW are in Figs. 9(c and d). The emulative wall formed smaller gaps than the hybrid walls because of increased (but still small) wall panel shear deformations and, to a greater extent, the large base slip that contributed to the total wall displacements.

By using the 3D-DIC data for the relative vertical displacements across the base joint, the neutral axis (i.e., contact) length was determined. Fig. 9(e) plots the measured and predicted neutral axis length at the south end of Specimens HW2 and HW3. Similar results for Specimen EW are depicted in Fig. 9(f). The results are shown for the peak of the first cycle in each drift series, except for the last series where all three cycles are shown. During the relatively small displacements of each wall, the neutral axis length went through a rapid decrease associated with gap opening at the base. Due to the compression forces from the PT steel, the neutral axis length was larger in the hybrid walls compared to the emulative wall during the early cycles. As each wall was displaced further, the neutral axis length continued to decrease but at a much slower pace. Once deterioration of the concrete at the wall toes initiated, the neutral axis length began to elongate to satisfy equilibrium with the reduced concrete stresses. This effect is particularly evident for Specimen HW3 (due to crushing of the confined concrete) and during the final drift series for Specimen EW (due to concrete splitting). The
elongation of the neutral axis did not occur in Specimen HW2 because damage to the concrete was small due to the pullout of the ED bars (i.e., smaller tension forces in the ED steel led to smaller concrete compressive stresses at the wall toes).

The analytical models generally predicted a smaller neutral axis length, particularly during the small displacement cycles, which could be related to the inability of the models to accurately capture the concrete compression stresses at the wall toes as gap opening occurred. The model for Specimen HW3 underpredicted the elongation of the neutral axis length under increased confined concrete damage during the larger displacement cycles for the structure. The discrepancy in Specimen HW2 was due to the premature pullout of the ED bars, which was not included in the analysis.

**Wall Uplift**

The vertical displacement at the top of the wall, which is related to the gap opening at the base joint, was used to determine the residual uplift upon unloading (i.e., residual axial heightening of the structure). Fig. 10(a) shows the residual uplift measured at the centerline of each wall at the location of the applied lateral load after unloading to $\Delta_w = 0\%$ from the third cycle in each drift series. The
accumulation of this residual uplift provides a measure of the axial restoring force in the structure.

In Specimens HW2 and HW3, the residual uplift did not begin to accumulate until the $\Delta_w = \pm 1.55\%$ cycles, which coincided with the initiation of the PT stress losses. Of these two hybrid walls, Specimen HW3 had a larger residual uplift with a maximum of 0.25 cm (0.10 in.) upon unloading from $\Delta_w = \pm 2.95\%$ (the smaller uplift in Specimen HW2 would be expected because of the pullout of the ED bars). The small amount of uplift in the hybrid walls did not affect the performance of these walls in any undesirable way. In contrast, Specimen EW accumulated significantly greater residual uplift beginning from the $\Delta_w = \pm 0.27\%$ cycles. Over successive loading/unloading cycles with increasing drift, the residual (plastic) tensile elongations (strains) in the ED bars accumulated rapidly, causing a maximum residual wall uplift of 0.61 cm (0.24 in.) after the last cycle to $\Delta_w = \pm 1.15\%$ [Fig. 10(a)]. The large residual uplift of the emulative wall was due to the lack of an adequate restoring force (provided only by the gravity load) to yield the ED bars back in compression. The complete uplift of the wall at the base joint led to in-plane shear-slip failure with large strength and stiffness degradation.

**Slip at Horizontal Joints**

The horizontal slip at the upper joint of each wall was negligible. For the base joint, the solid and dashed lines in Fig. 10(b) show the measured slip at the centerline of the walls for the peak of the third cycle in each drift series (except for the final drift series where both the first and third cycles are plotted). The maximum slip at the base of Specimen HW2 [approximately 0.25 cm (0.10 in.) at $\Delta_w = \pm 2.30\%$ and 0.38 cm (0.15 in.) at $\Delta_w = \pm 2.30\%$] and Specimen HW3 [approximately 0.12 cm (0.05 in.) at $\Delta_w = \pm 2.95\%$ and 0.31 cm (0.12 in.) at $\Delta_w = \pm 2.95\%$] was small. Further, crushing of the concrete at the wall toes did not result in a disproportionate accumulation of slip.

For the hybrid walls, the base slip toward the end of each test exceeded the allowable slip of 0.15 cm (0.06 in.) per ACI ITG-5.1-07 [shaded region in Fig. 10(b)]. However, the performance of the walls was not negatively affected by the slip; thus, the current slip limit may be too conservative. Based on these results, a more reasonable slip limit may be 0.38 cm (0.15 in.). For Specimen EW, the base slip was much larger [with a maximum of 1.3 and 2.4 cm (0.5 and 0.9 in.) at the third cycle of $\Delta_w = \pm 1.15\%$ and $\pm 1.5\%$, respectively]. The slip accumulated rapidly beginning at $\Delta_w = \pm 0.27\%$ when, as shown in Fig. 10(a), residual wall uplift also initiated. Upon unloading from complete uplift at the base, the ED bars were the only components transferring shear from the wall into the foundation until the wall drift was large enough to close the gap at the compression toe of the wall. The concrete around the ED bars deteriorated because of the shear transfer, ultimately causing splitting of the base panel around the bars [Fig. 6(c)]. At $\Delta_w = \pm 0.27\%$, when base slip started to rapidly accumulate in Specimen EW, the coefficient of shear friction was calculated as 0.41 by dividing the measured base shear with the estimated compression force transferred through the contact region. This estimated friction coefficient is considerably lower than the value of 0.50 specified by ACI ITG-5.2-09 for the base joint.

**Summary and Conclusions**

This paper investigates the potential for the use of precast walls as special RC shear walls in high seismic regions. The following conclusions are made from the research:

1. ACI 318-11 Type II mechanical splices are not recommended for the ED bars. Instead, continuous bars that are fully developed on both sides of the base joint should be used.

2. The lack of a PT force in the emulative wall specimen resulted in inadequate restoring, leading to excessive uplift, horizontal slip, and lateral strength and stiffness degradation. These walls are not recommended for seismic regions unless a reliable amount of tributary gravity load exists to fully yield the tensile ED bars back in compression upon unloading.

3. The hybrid wall specimens behaved essentially as a rigid body dominated by gap opening at the base joint, with limited concrete cracking in the base panel. In comparison, cracking in the emulative specimen was more extensive and spread high into the upper panel.

4. The shear-slip limit in ACI ITG-5.1-07 can be increased to 0.38 cm (0.15 in.) without negatively affecting the wall performance. The crushing of the concrete at the wall toes did not result in a disproportionate accumulation of slip.

5. The first confinement hoop at the wall toes should be placed as close to the bottom of the base panel as permitted by the cover requirements of ACI 318-11. Furthermore, the length-to-width aspect ratio of the hoops should be limited and crossties not directly engaging the hoop steel should not be considered as an effective component of the confinement reinforcement.

6. As required by ACI 318-11, the horizontal distributed bars should be embedded into the confinement cage at the toes of the base panel to maintain the effectiveness of these bars and minimize the extent of the concrete damage (i.e., concrete cracking, cover concrete spalling, and bar delamination).

7. Fiber-element analytical models of the walls were able to replicate the overall base shear versus lateral displacement history, gap opening/closing behavior at the base joint, and the behavior of the PT steel and ED bar reinforcement.
Recognitions

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