Performance of Wood-Frame Wall with Thin ECC Shear Panel

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Abstract: The study evaluates an alternative to traditional wood-framed shear wall construction, an innovative use of a water and seismic damage resistant, wood-concrete-composite (WCC). The WCC design consisted of a thin layer of engineered cementitious composite (ECC) cast in a traditional wood frame. Structural performance and damage sustained during lateral loading and cost were evaluated, and compared to a traditional wood frame with OSB sheathing. Monotonic tests of WCC walls show that the average maximum load was 47.5 kN (10,700 lb), average elastic shear stiffness was 1.78 kN/mm (10,200 lb/in.) and average energy absorbed was 4,810 J (42,600 lb-in.). Tests show WCC is comparable with or superior to the OSB wall in shear strength and stiffness, energy absorption and ductility. During lateral loading the WCC appeared to sustain less damage than the OSB wall. Panelized construction of the WCC may increase cost but yield benefits such as decreased construction time and greater durability. WCC appears to be a viable shear wall system that should be refined and tested for building code compliance.

DOI: 10.1061/(ASCE)1084-0680(2009)14:3(123)

CE Database subject headings: Wood structures; Shear walls; Full-scale tests; Concrete structures; Composite structures; Frames; Walls.

Introduction

This project seeks a solution to performance problems associated with seismic damage and moisture damage sustained by traditional wood-framed shear walls, using an innovative wood-concrete-composite (WCC). The WCC design consists of a thin layer of engineered cementitious composite (ECC) cast in a traditional wood frame.

Costs of repairs and damage to wood-framed structures after some seismic events have been enormous. The 1994 Northridge, California earthquake caused approximately $20 billion in structural and nonstructural losses to wood-framed buildings (Kircher et al. 1997). Additionally, “48,000 units, almost all of them in wood-frame buildings, were rendered uninhabitable by the earthquake” (Seible et al. 1999). More damage resistant structures may save billions of dollars in future seismic repairs, provide greater safety and reduce the number of displaced inhabitants.

Data from Oregon indicate that construction defect claims are widespread and mainly from water intrusion into the building envelope. Approximately 15% of new homes in Oregon have at least one major construction defect. In 84 percent of the cases, water intrusion is the cause. The location of the defect is the building envelope in 78% of the cases (AACG 2006). Prolonged wet conditions in all-wood construction allow for mold and decay fungi growths. Water intrusion can cause mold; a human health hazard, and decay fungi; a structural performance hazard. Using a high performance concrete panel instead of oriented strand board (OSB) or plywood, the surface area for mold growth is significantly reduced as is the food source for mold and decay fungi.

Current wood shear wall designs can be badly damaged during earthquakes (Filiatrault 2001). Wood shear walls are damaged by inelastic deformation while dissipating seismic forces. Energy dissipation can provide a substantial level of damping to a structure; restricting deflection and damage to a manageable level (Naeim 2001). During cyclic lateral loading, typical wood framed construction has three energy dissipation modes: inelastic nail deformation (flexure), friction between components (at connections of nails, sheathing and studs) and crushing of the wood. Crushing occurs in the initial loading cycles, pinching the hysteretic response during secondary cycles of equal or less displacement. Wood crushing reduces the stiffness and energy dissipation potential of the shear wall during secondary cycles. Nail deformation and crushing damage the wall and contribute to high repair costs for wood framed construction (Dinehart et al. 2006)

Energy dissipation modes for the WCC construction are fundamentally different from those of wood sheathed walls. WCC walls dissipate energy through deformation of the polymer reinforcement. The structure is designed to prevent fasterener (staple) deformation or withdrawal and damage to the wood frame. The wall is designed so the reinforcement strength is less than the fasterener yield or withdrawal strength and wood strength.

Concrete materials used in these walls were ECC. This is a fiber-reinforced concrete-like material that exhibits strain hardening and steady-state, flat cracking behavior in tension and flexure. Steady-state, flat cracking occurs when each crack that initiates in the concrete matrix maintains a uniform and very small width along its entire length. Fiber bridging allows the fibers in the concrete to continue transferring load across the crack without...
loss of strength. Polyvinyl alcohol (PVA) fibers are used in this variety of ECC to provide tensile strength during fiber bridging. If the ECC should be exposed to tensile stress that causes cracking, the behavior of the material is ductile. The PVA-ECC used in this study is usually ductile to approximately 2–5% tensile strain. (Li 2003). ECC also provides an extremely durable shell that can withstand severe freeze/thaw cycles and has low permeability to moisture.

Objectives

1. To design, build and test a shear wall design that is damage resistant and economical using a wood-concrete-composite. Determine the shear strength, shear stiffness, energy absorption, ductility, and damage level of the WCC wall system under lateral loading.

2. Evaluate and compare the performance of the WCC shear wall to traditional wood-frame construction of nearly equal shear strength in terms of structural performance, damage sustained under racking loads and cost effectiveness.

Literature Review

Bathon et al. (2006) studied a wood-concrete-composite design of concrete panels attached to timber framing. Advantages of combining wood and concrete for residential buildings are: (1) wood framing allows construction of relatively light and open structures; (2) wood framing reduces dead load; and (3) concrete provides fire resistance and greater stiffness for wind and seismic loads. WCC was studied to examine its resistance to hurricane and seismic loadings. The wood-to-concrete shear connection (for floors, walls, and roofs) was accomplished through a single row of continuous steel shear connectors. Performance of any composite system depends greatly on the shear connection. The continuous shear connector creates a stiff but ductile connection system. The flat steel rib of the connector is 80 mm tall with the tin and the upper half cast in concrete. Under monotonic loading, testing showed this continuous shear connection creates a stiffer and stronger composite action compared to mechanical connections (nails, bolts, or screws). Strength of the connection was approximately 60 percent greater than traditional mechanical connections. Bathon et al. (2006) concludes that WCC is a cost-effective hurricane and seismic resistant design for residential buildings.

The present study, however, is unique and different from Bathon et al. (2006) in the following ways:

1. Here, the concrete layer is approximately 12 mm (0.5 in.) in thickness, while Bathon et al. (2006) used concrete thicknesses of approximately 75 mm (3.0 in.). The 85% thinner concrete layer yields significant reductions in material costs and weight.

2. For this WCC system, a polymer grid fastens the ductile concrete layer to the traditional light wood frame, and forms the primary failure mechanism for the wall. None of the literature showed a similar overall system, fastening method or failure mechanism.

Materials and Methods

Wall Specimens

Racking load test specimens measured 2440 mm × 2440 mm (Fig. 1). Wall design was identical for each specimen, and tested in accordance with ASTM E 564 (2006a) and/or ASTM E 2126 (2005) CUREE protocol requirements. The G-Series wall design was selected based on overall performance from tests designated A-Series through G-Series shown in Lewis (2008).

All frames were constructed of #2 and better Douglas Fir-Larch 38 mm × 89 mm (2 in.× 4 in. nominal) kiln dried lumber. Studs were sistered for the top plate, two end posts and sill plate. Vertical studs were spaced at 406 mm (16.0 in.) on center and connected to the top and bottom plate by two 4.8 mm diameter × 89.0 mm long (0.190 in. diameter × 3.5 in. long) galvanized screws per connection. A Senco SN 65 pneumatic nail gun was used to connect the sistered studs with two 3.33 mm diameter × 82.6 mm long (16 d) galvanized nails per connection at 203 mm (8.0 in) on center. Each corner of the stud frame was reinforced with a 1.22 mm (18 gauge) thick sheet metal L bracket. Each bracket leg measured 254 mm long × 75 m wide (10 in long × 3.0 in. wide) and was secured to the stud frame using ten 4.2 mm diameter × 76.2 mm long (0.164 in. diameter × 3.0 in. long) galvanized deck screws per leg. Seismic hold-downs were Simpson Strong-Tie HDQ8 (Simpson Strong-tie, Pleasanton, Calif.) with twenty SDS 6.4 mm diameter × 76.2 mm long (0.25 in. diameter × 3.0 in. long) screws each. Each hold-down was attached to an end post and secured to the lower rail of the test fixture with one 15.9 mm (0.625 in.) diameter, Grade 5 steel all-thread rod. Anchorage for shear was provided by two 15.9 mm (0.625 in.) diameter, Grade 5 steel bolts. Bolts were installed with a washer and a 51.4 mm × 51.4 mm × 63.5 mm (2.0 in. × 2.0 in. × 0.25 in.) SIMPSON BP5/8-2 base plate between the head of the bolt and wood sill plate.

The stud frame was covered with a 3050 mm × 3050 mm × 0.17 mm sheet of polyethylene house wrap (Tyvek). House wrap was stapled to the stud framing with a DUO-FAST ANC 5418 pneumatic stapler (ITW Industrial Fastening, Elgin, Ill.) using 14.3 mm long × 4.8 mm crown × 20 gauge wire (0.56 in. long × 0.19 in. crown × 20 gauge wire) staples at approximately 460 mm (18.0 in.) O.C. It serves as a water barrier and as lost formwork for the ECC.

Polymer grid material known as Mirafl BXG 11 geogrid was placed over the house wrap and stapled with a Bostitch 650S4 pneumatic stapler using 38.1 mm long × 12.7 mm crown × 16 gauge wire (1.5 in. long × 0.5 in. crown × 16 gauge wire) staples at 64 mm (2.5 in.) O.C. to each stud in the frame. A friction joint was created by “sandwiching” the reinforcement between the sistered framing members around all four edges of the frame. Grid strands were aligned with the direction of the framing members, designated as “0°.” Rigid foam backing was placed into each stud bay to help support the weight of the ECC during pouring. The frame assembly was placed in a horizontal position and a 13 mm (0.5 in.) layer of ECC was poured on to the frame as-
assembly, then troweled and screeded flat. The ECC flowed around and encapsulated the polymer grid except at each grid/staple connection. Each shear wall was cured for approximately 28 days before testing.

**ECC Specimens**

One ECC compression cylinder was created for each wall that was tested. One ECC uniaxial tension strength specimen was tested for walls 1G1 and 1G2. Each compression test cylinder measured 203 mm long × 102 mm diameter (8.0 in. long × 4.0 in. diameter). Tension test coupons measured 13 mm × 76 mm × 203 mm (0.5 in. × 3.0 in. × 8.0 in.). All ECC specimens were cured for approximately 28 days before testing. Table 1 provides a typical ECC mix design.

**Wall Test Setup**

A hydraulic actuator with a 489 kN (110 kip) capacity and a 254 mm (10.0 in.) stroke was used to apply loads to the frame. The actuator was attached to the flange of a W 10 × 112 steel beam that was mounted vertically to a concrete reaction wall. Another 102 mm (4.0 in.) hydraulic cylinder was mounted vertically to support the weight of the actuator so no vertical load was placed on the shear wall during testing. A 2.7 m (9.0 ft) long, welded steel top rail was attached to the actuator and mounted to the top of the shear wall by using six 12.7 mm (0.5 in.) diameter steel all-thread rods with a 51.4 mm × 51.4 mm × 6.35 mm (2.0 in. × 2.0 in. × 0.25 in.) SIMPSON BP5/8-2 base plate and washer placed between the nut and the wood top plate. The steel top rail was also attached to a lateral brace that prevented any out-of-plane movement of the shear wall during testing. The base of the shear wall was secured (using hold-downs and anchor bolts described previously) to a welded steel fixture that was firmly attached to the concrete and steel I-joint composite test floor.

Hydraulic actuator position was regulated with an MTS 407 controller (MTS Systems Corporation, Eden Prairie, Minn.). Load and deflection data were gathered from the actuator’s internal load and position sensors, and also from four linear variable differential transformers (LVDT’s). One LVDT was located at each lower corner of the wall to measure uplift, one LVDT measured base slip and one LVDT measured top plate slip. Data were acquired using a personal computer running LabVIEW Version 7.1 (National Instruments Corporation, Austin, Tex.).

**ECC Test Setup**

Properties of ECC samples were determined by concrete compressive test ASTM C 39 (2006b) and a uniaxial tensile strength test. Tensile testing was performed by the Advanced Civil Engineering Materials Research Lab (ACEMRL), University of Michigan, Ann Arbor, Mich., in accordance with the ACEMRL standard. Compressive tests to failure used a Tinius-Olsen Concrete Compression Tester model Super “L” at a rate of 241 kPa/s (35.0 psi/s).

### Table 1. Typical ECC Mix Design (Data from Li and Lepech 2004)

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity (kg/m³)</th>
<th>Weight percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland cement (Type I)</td>
<td>583</td>
<td>27.9%</td>
</tr>
<tr>
<td>Silica sand</td>
<td>467</td>
<td>22.3%</td>
</tr>
<tr>
<td>Fly ash (Type F)</td>
<td>700</td>
<td>33.5%</td>
</tr>
<tr>
<td>Water</td>
<td>298</td>
<td>14.2%</td>
</tr>
<tr>
<td>Commercially available superplasticizer</td>
<td>17.5</td>
<td>0.8%</td>
</tr>
<tr>
<td>PVA fiber</td>
<td>26.0</td>
<td>1.2%</td>
</tr>
</tbody>
</table>

**Loading Protocol**

This study used two types of shear wall racking tests: monotonic (ASTM E 564-06) and cyclic (ASTM E 2126-05 CUREE protocol). Monotonic test results also established the reference deformation (Δref) needed for cyclic testing. The ASTM E 564 test procedure is recommended for determining the performance of the entire wall assembly (Filiatrault 2001).

**Monotonic Protocol**

Monotonic tests were performed on three walls of the G-series configuration for racking resistance per ASTM E 564-06. This racking test applies a unidirectional concentrated horizontal load at the upper edge of the wall, in the plane of the shear wall, while the base of the wall is held fixed. Walls 1G1 and 1G2 were loaded at a constant displacement rate of 0.13 mm (0.005 in./s) to a total displacement of 127 mm (5.0 in.). Wall 1G3 was loaded at a constant displacement rate of 0.13 mm/s (0.005 in./s) to a total displacement of 140 mm (5.5 in.) to be sure of reaching the ultimate displacement (Δu). Data were gathered at 20 Hz.

**CUREE Protocol**

The CUREE protocol for ordinary ground motions was used for the cyclic portion of the testing. This racking test applies a cyclic (reversed) lateral load at the upper edge of the wall, in the plane of the shear wall, while the base of the wall is held fixed. Displacement amplitudes were scaled by the reference displacement (Δref) from the monotonic tests. The CUREE protocol begins with small amplitude cycles known as initiation cycles. The protocol then introduces single occurrence larger amplitude cycles known as primary cycles. Between each primary cycle are a number of cycles that are 75% of the amplitude of the preceding primary cycle, known as trailing cycles. The sequence of the cycles varies in length from three to seven cycles with primary cycles increasing in amplitude as the test progresses. The hydraulic actuator was positioned at mid-stroke at the beginning of each test. This allowed a maximum actuator displacement of 127 mm (5.0 in.) in both directions. CUREE test walls were loading at a constant cyclic frequency of 0.25 Hz and data recorded at 20 Hz.

**Data Analysis**

Wherever possible, for standardization, terminology from ASTM E 2126-05 is used.

**Monotonic Tests**

Data from the monotonic tests were used to directly generate load–deflection curves for each of the shear walls 1G1, 1G2 and 1G3. Specific points of interest from the load–deflection curve include: maximum load (Ppeak); deflection at peak load (Δpeak); ultimate displacement (Δu); elastic shear stiffness (K); absorbed energy (E) up to ultimate displacement; and the reference displacement (Δref). The ultimate displacement (Δu) is the first occurrence postpeak of the load dropping to 0.8Ppeak. Ultimate displacement (Δu) is also known as the failure limit state. Reference displacement (Δref) was calculated as 60 percent of ultimate displacement (Δu), and used in the cyclic test to scale the amplitude of the CUREE protocol displacements (Fig. 2).
Elastic shear stiffness \( (K_e) \) was determined at 40% of the maximum load as follows:

\[
K_e = \frac{0.4P_{\text{peak}}}{\Delta_{g0.4}}
\]

Absorbed energy \( (E) \) was used as an alternative to ductility based on yield displacement, and is the area under the load-displacement curve up to the ultimate displacement \( (\Delta_u) \).

**CUREE Tests**

The load-deflection data obtained from the cyclic tests were used to generate hysteretic curves for each wall specimen. An envelope curve was generated using the peak load of each primary cycle during the CUREE protocol. Positive and negative envelope curves were generated for each wall specimen and results averaged per ASTM E 2126. Specific points of interest from the envelope curve include: maximum load \( (P_{\text{peak}}) \); deflection at peak load \( (\Delta_{\text{peak}}) \); ultimate displacement \( (\Delta_u) \); elastic shear stiffness \( (K_e) \); and absorbed energy \( (E) \) up to ultimate displacement. Values for maximum load \( (P_{\text{peak}}) \), elastic shear stiffness \( (K_e) \), and absorbed energy \( (E) \) were calculated as described above for the monotonic curve (Fig. 2). The equivalent energy elastic-plastic curve (EEEP) and ductility ratio were also calculated per ASTM E 2126.

**m-Factor Analysis**

ASCE 41 (2006) defines an \( m \)-factor, that is always greater than or equal to unity and is the, “component demand modifier factor to account for expected ductility associated with this action at the selected structural performance level” (ASCE 2006). \( m \) is used to modify the expected structural strength of the component as follows:

\[
m \cdot \kappa \cdot Q_{CE} \geq Q_{UD}
\]

Eq. (1) defines the acceptance criteria for deformation-controlled structures such as shear walls; “\( \kappa \)"=knowledge factor that accounts for uncertainty when analyzing an existing structure; “\( Q_{CE} \)”=expected strength of the component at the deformation level under consideration; and “\( Q_{UD} \)”=total force applied to the component due to gravity and earthquake loads. The \( m \)-factor alone should not be used as a direct performance indicator. The expected strength \( (Q_{CE}) \) must also be considered for evaluating performance (ASCE 2006).

The \( m \)-factor is calculated by analysis of an idealized load-displacement curve that is superimposed over actual test data (Fig. 3). The idealized load-displacement curve is generated by extending a secant stiffness line that intersects the actual test data curve at 60 percent of peak load, thus creating line segment AB. Line segment BC is then drawn using a small percentage (0–10%) of the slope of line AB (a slope of 5% of line AB was used in this analysis). The abcissa of point C is equal to the ultimate displacement \( (\Delta_u) \) of the particular test specimen under consideration. Point C is defined as the failure point of the component. The length of line segment AB is adjusted to create an area under the idealized load-displacement curve equal to the area under the actual test data curve. The area under the curve is the energy absorbed \( (E) \). Point B is the yield point of the component.

There are three structural performance levels defined in ASCE 41: immediate occupancy (IO), life safety (LS), and collapse prevention (CP). The \( m \)-factors for each performance level are calculated as the ratio of the drift for the desired performance level to the drift at the yield point. The \( m \)-factor for CP is the ratio of the drift at the failure point (point C) to the drift at the yield point (point B) multiplied by a factor of 0.75. The \( m \)-factor for LS is the ratio of 75% of the drift at the failure point (point C) to the drift at the yield point (point B) multiplied by a factor of 0.75. The \( m \)-factor for IO is the ratio of 50.2% of the drift at the failure point (point C) to the drift at the yield point (point B) multiplied by a factor of 0.75 per ASCE 41-06 section 2.8.3.7.

**Results and Discussion**

**Monotonic Tests**

Three ECC shear walls (1G1, 1G2, and 1G3) were tested in accordance with ASTM E 564-06. The load-displacement curves for these walls are presented in Fig. 4 and results for monotonic tests shown in Table 2. Data from these tests were used to determine maximum load \( (P_{\text{peak}}) \), unit shear strength \( (v_{\text{peak}}) \), elastic shear stiffness \( (K_e) \), deflection at peak load \( (\Delta_{\text{peak}}) \), ultimate deflection \( (\Delta_u) \), absorbed energy \( (E) \) up to ultimate deflection, and reference displacement \( (\Delta_{\text{ref}}) \).

Initially, two walls (1G1 and 1G2) were tested for unit shear strength \( (v_{\text{peak}}) \) and shear stiffness \( (K_e) \). Shear stiffness was not within 15 percent as required by ASTM E 564, therefore, a third wall (1G3) was tested. After the third test, strength and stiffness values were taken as the average of the two weakest specimens, 1G1 and 1G2. Average strength and stiffness values for 1G1 and 1G2 were 47.1 kN (10,600 lb) and 1.84 kN/mm (10,500 lb/in) respectively. Load–displacement curves in Fig. 4 show fairly consistent behavior up to ultimate displacement \( (\Delta_u) \) except as noted: wall 1G2 showed higher initial stiffness than the other two walls, and wall 1G1 showed a drop in load carrying capacity at approxi-
Fig. 4. G-Series monotonic load-displacement curves

Table 2. ECC and Langlois (2002) OSB Monotonic Data Comparison

<table>
<thead>
<tr>
<th>Panel type</th>
<th>Test</th>
<th>$P_{\text{peak}}$ kN (lb)</th>
<th>$K_e$ kN/mm (lb/ft)</th>
<th>$E$ Joules (lb-in.)</th>
<th>$\Delta_u$ mm (in.)</th>
<th>$\Delta_{\text{peak}}$ mm (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OSB</td>
<td>Mono 1</td>
<td>41.4</td>
<td>9,320</td>
<td>13,000</td>
<td>2,400</td>
<td>21,300</td>
</tr>
<tr>
<td>OSB</td>
<td>Mono 2</td>
<td>40.0</td>
<td>9,000</td>
<td>11,900</td>
<td>3,040</td>
<td>26,900</td>
</tr>
<tr>
<td>OSB</td>
<td>Mono 3</td>
<td>42.9</td>
<td>9,650</td>
<td>8,800</td>
<td>4,100</td>
<td>36,300</td>
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<tr>
<td></td>
<td>Avg</td>
<td>41.4</td>
<td>9,323</td>
<td>11,233</td>
<td>3,180</td>
<td>28,167</td>
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<tr>
<td>COV</td>
<td></td>
<td>3.50%</td>
<td>19.3%</td>
<td></td>
<td>27.0%</td>
<td>12.0%</td>
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<td>ECC</td>
<td>1G1</td>
<td>48.1</td>
<td>10,800</td>
<td>8,630</td>
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<td>12,300</td>
<td>4,820</td>
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<td>-9.5%</td>
<td>51.4%</td>
<td>2.8%</td>
<td>24.6%</td>
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</table>

Three ECC shear walls (2G1, 2G2, and 2G3) were tested in accordance with ASTM E 2126-05. A reference displacement ($\Delta_{\text{ref}}$) of 76 mm (3.0 in.) was used in all cyclic tests. Walls 2G1 and 2G2 were tested and the strength and stiffness values were found to be within 10 percent of each other. The averages of these two tests were used to determine the cyclic strength and stiffness of the G-Series walls. A third cyclic test was not required to determine average strength and stiffness values per ASTM E 2126. This allowed the third wall to be tested for damage accumulation using a segmented cyclic test. Data collected from these tests were used to determine maximum load ($P_{\text{peak}}$), deflection at peak load ($\Delta_{\text{peak}}$), ultimate displacement ($\Delta_u$), elastic shear stiffness ($K_e$), and absorbed energy ($E$) up to ultimate displacement ($\Delta_u$).

**ECC Tests**

Compressive strength tests were conducted for each ECC panel. The average ECC compressive strength was approximately 82,700 kPa (12,000 psi), indicating a high strength concrete. The typical strength of concrete is up to approximately 41,400 kPa (6000 psi) (Kosmatka et al. 2002). The density of the ECC mix was 2192 kg/m³ (137 lb/ft³).

Uniaxial tension tests were performed for the ECC used in walls 1G1 and 1G2. Both specimens exhibited tensile strain capacity in excess of 1.5 percent, reflecting ECC’s ductility.

Lepech and Li (2006) tested ECC and normal roadway concrete for freeze-thaw resistance per ASTM C 666 (2003). Their findings indicate that ECC is approximately ten times more durable to freeze-thaw conditions than normal roadway concrete. Dynamic modulus and strain capacity of the ECC specimens showed no significant change, even after 300 cycles over 14 weeks of testing. Additionally, ECC was subjected to accelerated weathering tests showing that over the equivalent of a 70-year period of hot and humid conditions, the ductility was reduced from 4.5 to 2.8%, and still acceptable for nearly any application. Further, the water permeability coefficient of ECC was shown to be approximately $1.0 \times 10^{-10}$ m/s.

ECC properties include high ductility, compressive strength, durability and resistance to weathering. All make it an ideal material for use as a shear panel and an exposed exterior wall.

**Performance Comparison**

WCC results are compared to an all-wood shear wall study by Langlois (2002) using the same monotonic and cyclic tests. Langlois’ (2002) shear wall was constructed of traditional wood
Table 3. ECC and Langlois (2002) OSB Cyclic Data Comparison

<table>
<thead>
<tr>
<th>Panel type</th>
<th>Test</th>
<th>$P_{peak}$ kN</th>
<th>(lb)</th>
<th>$K_e$ kN/mm</th>
<th>$E$ Joules</th>
<th>$\Delta_{peak}$ mm</th>
<th>(in.)</th>
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<tr>
<td>OSB</td>
<td>CUREE 4.1</td>
<td>36.0</td>
<td>8,100</td>
<td>1.80</td>
<td>10,300</td>
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<tr>
<td>OSB</td>
<td>CUREE 4.2</td>
<td>37.9</td>
<td>8,530</td>
<td>2.40</td>
<td>13,700</td>
<td>1,970</td>
<td>17,500</td>
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<tr>
<td>Avg</td>
<td></td>
<td>37.0</td>
<td>8,315</td>
<td>2.10</td>
<td>12,000</td>
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<td>19,050</td>
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<td>ECC</td>
<td>2G1</td>
<td>44.5</td>
<td>10,000</td>
<td>2.14</td>
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<td>2,960</td>
<td>26,200</td>
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<td>ECC</td>
<td>2G2</td>
<td>45.4</td>
<td>10,200</td>
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<tr>
<td>Avg</td>
<td></td>
<td>44.9</td>
<td>10,100</td>
<td>2.07</td>
<td>11,800</td>
<td>2,870</td>
<td>25,400</td>
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<td>Difference (%)</td>
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<td>21.6%</td>
<td>-1.6%</td>
<td>33.5%</td>
<td>-18.0%</td>
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</tbody>
</table>

38 mm × 89 mm (2 in. × 4 in. nominal) studs, spaced at 406 mm (16 in.) O.C. Framing was covered with 11.9 mm (0.5 in.) thick OSB sheathing nailed at 102 mm (4.0 in.) edge spacing and 305 mm (12 in.) field spacing with 3.33 mm diameter × 82.6 mm long (8 d) ring shank nails.

Monotonic Tests
ECC and OSB walls were compared with respect to lateral loading (monotonic and cyclic) performance, damage sustained during loading, raw material cost and total project cost.

An ECC and OSB monotonic load-displacement data comparison is shown Table 2. Three parameters were used for the comparison: maximum load ($P_{peak}$), elastic shear stiffness ($K_e$), and absorbed energy ($E$) up to the ultimate displacement ($\Delta_u$). Average ECC wall maximum load was 14.8% higher than the OSB wall. This parameter was reasonably similar between the two wall systems, and allows for a meaningful system comparison. ECC monotonic curves were much more uniform than the OSB monotonic curves through the full range of deflection. Results of ECC and OSB monotonic tests indicate that the ECC system is 9.4% less stiff, but absorbed 51.3% more energy than the OSB system.

CUREE Tests
The ECC cyclic envelope curve comparisons with OSB are shown in Table 3. Three parameters were used for the comparison: maximum load ($P_{peak}$), elastic shear stiffness ($K_e$), and absorbed energy ($E$) up to ultimate displacement ($\Delta_u$).

Results of the ECC and OSB cyclic test results indicate the ECC system had a 22.0% higher peak load, 1.4% lower stiffness and absorbed 33.5% more energy than the OSB system. Energy absorbed by the ECC system was significantly higher than the OSB system. ECC peak load was 14.8% (monotonic test) to 22.0% (cyclic test) higher than the OSB peak load.

$m$-Factor Analysis
An $m$-factor (ductility) analysis was performed for each of the monotonic tests and compared to Langlois (2002). Since expected strength ($Q_{CE}$) must also be considered for evaluating performance, Langlois' (2002) walls of approximately equal strength were used in the comparison. Average values for ECC and OSB walls are shown in Table 4. The only $m$-factor that was greater than the value given by ASCE 41 was $m_{IO}$ of the ECC wall. ECC $m$-factor values were greater than the Langlois (2002) OSB $m$-factor values by approximately 43%. The results indicate the ductility of the ECC wall is much greater than the OSB wall of Langlois (2002).

Limitations
The present study should be considered as a first step in the evaluation process of the WCC wall system. More lateral loading tests should be performed to provide better statistical data. The conclusions in this study are based on the limited number of tests performed and could be modified based on additional testing. Further, the “Cost Comparison” section is based on preliminary and limited information and subject to change as more information is available.

Also, other factors relating to the seismic performance of WCC walls should be studied in more detail. For example, the natural period of any structural system is determined by its mass and stiffness. Weight of each 2440 mm × 2440 mm WCC wall section is approximately 2.2 kN (490 lb) compared to approximately 1.6 kN (360 lb) for an equal size all-wood wall with siding. Stiffness of the WCC wall system is, on average, greater than Langlois’ (2002) walls. Mass and stiffness of the WCC walls are different from the all-wood wall system, thus affecting the seismic response. Mass of the lateral system also includes the roof, therefore the period of the WCC system could be greater or less than an all-wood shear wall system depending on the configuration of the specific building.

Cost Comparison
In practice, WCC walls could be constructed as a series of panelized units assembled to form the finished exterior of a house. Units could be fabricated inside a climate-controlled factory and shipped to the building site. Panels could be erected using a small crane and securely fastened together using bolted connections and sealant at each panel joint. In contrast, all-wood structures with OSB shear panels are typically built on-site by a contractor with a framing crew.

Cost estimates are based on the following assumptions: WCC panelized units are comparable to structural insulated panel (SIP) units in unit sales price and total cost of the finished structure.

Table 4. $m$-Factor (Ductility) Comparison for Monotonic Test

<table>
<thead>
<tr>
<th></th>
<th>ECC average values</th>
<th>Langlois (2002) average values</th>
<th>ACSE 41 (Table 8–3) for wood shear walls</th>
</tr>
</thead>
<tbody>
<tr>
<td>$m_{CF}$</td>
<td>3.60</td>
<td>2.52</td>
<td>4.50</td>
</tr>
<tr>
<td>$m_{LS}$</td>
<td>2.70</td>
<td>1.89</td>
<td>3.80</td>
</tr>
<tr>
<td>$m_{IO}$</td>
<td>1.81</td>
<td>1.27</td>
<td>1.70</td>
</tr>
</tbody>
</table>

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Total cost estimates of the completed house are based on local (Pacific Northwest) labor rates, and local retail material cost data from March 2008 (Lewis 2008). Raw material cost includes such items as wood framing, fasteners, ECC components and sheathing. Sales price is raw material cost multiplied by a mark-up factor, to cover costs associated with fabrication.

Unit sales price for SIP’s range up to $5.58 per square foot. Unit raw material cost of WCC walls is approximately $1.72 per square foot. WCC cost is approximately 5% greater than the OSB walls of Langlois (2002). The WCC walls had much less damage to framing and sheathing. WCC walls carried an average maximum load of 45.1 kN (10,100 lb) with an average elastic shear stiffness of 2.07 kN/mm (11,800 lb/ in.) and absorbed an average of 2870 J (10,200 lb s ft) with an average elastic shear stiffness of 2.70 in. m.

Conclusions

Monotonic tests showed WCC walls carried an average maximum load of 47.5 kN (10,700 lb) with an average elastic shear stiffness of 1.78 kN/mm (10,200 lb/ in.) and absorbed an average of 4,810 J (42,600 lb in.) of energy. Cyclic tests showed that WCC walls carried an average maximum load of 45.1 kN (10,100 lb) with an average elastic shear stiffness of 2.07 kN/mm (11,800 lb/ in.) and absorbed an average of 2870 J (25,400 lb in.) of energy. The m-factor (ductility) analysis provided the following m-factors: \(m_{CP}\) of 3.60, \(m_{LS}\) of 2.70, and \(m_{IO}\) of 1.81 for the WCC walls.

Monotonic and cyclic lateral loading tests indicate that WCC is comparable with or superior to the OSB wall in overall structural performance. Specifically, strength of WCC walls was higher, and energy absorption and ductility were much higher than Langlois’ (2002) all-wood wall. In comparison to Langlois’ (2002) all-wood walls, WCC shear walls appear to be more resistant to damage during lateral loading based on the damage accumulation test. Average strength and stiffness of the WCC walls were higher at each stage of loading in the cyclic damage accumulation testing. Strength and stiffness of WCC walls were much higher at the 3.1% (CP) drift level. Visual damage comparisons showed WCC walls had much less damage to framing and sheathing than the OSB walls of Langlois (2002) under the same loading conditions. Damage in WCC walls was generally confined to a small percentage of the reinforcing strands at grid/staple connections.

WCC shear walls have a cost that may be comparable to or slightly more than all-wood walls. However, other potential benefits include reduced construction time, decay fungi damage, and maintenance; and increased fire and pest resistance; and greater durability. Based on the performance, cost and damage level of the WCC wall system, it appears WCC shear walls may be a viable alternative to an all-wood shear wall system. It is recommended that the WCC shear wall design be refined by testing additional prototypes, and be subjected to the full range of certified tests required for building code acceptance. It is also recommended that a full-scale prototype house using the WCC technology be constructed to study the assembly method and performance of the WCC panelized system over a period of time in actual use. The WCC wall system in this paper is protected by U.S. Patent No. 12/156,175.

References


