Improving the Performance of Gypsum Wallboard in Wood Frame Shear Walls

Scott I. Goodall1 and Rakesh Gupta, M.ASCE2

Abstract: The overall goal of this project was to design a wood frame shear wall that could withstand greater displacement before damage occurred to the gypsum wall board (GWB). A total of 14 shear walls consisting of seven different designs with two walls built per design were tested to failure. Six of these walls had 1,105 × 610 mm window openings and eight did not. Shear wall test behavior generally agreed with the ASCE/SEI 41-06 performance-based drift criteria. The results of the shear wall visual failure comparison indicated that all innovative shear wall designs outperformed the control designs at 1% drift. This was because less GWB damage was observed in the innovative shear wall designs. At 2 and 3% drift, two designs performed worse than the control. Three designs performed superior to the control designs at 2 and 3% drift. The greater performance of all these designs can be attributed to the increase in strength and stiffness of these shear walls. However, superior performance of the two designs was attributable to the similar stiffness of both sides of the shear wall, resulting in equal load sharing and less damage to the GWB. DOI: 10.1061/(ASCE)CF.1943-5509.0000165, © 2011 American Society of Civil Engineers.

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Introduction

Eighty to 90 percent of all structures in the United States are wood frame (Malik 1995). An even higher percentage of residential structures use wood as the primary structural material. For example, in California, 99% of all residences are of woodframe construction (CUREE 1998). A residential structure is generally the greatest personal investment of anyone in the United States and represents a great economic value to society (McMullin and Merrick 2002). Protecting this investment from natural disasters such as earthquakes is an important task for engineers. Fortunately, wood generally performs very well during seismic events. The Northridge earthquake caused an estimated 20 billion dollars in damage to wood structures (Seible et al. 1999), but much of the damage occurred in nonstructural components such as gypsum wall board (GWB) (Schierle 2003), and little damage to wood structural members (solid wood studs and joists or oriented strand board and plywood sheathing).

Current building design codes do not account for the contribution of the nonstructural GWB into shear wall mechanical behavior. This was believed to be a conservative assumption until the wood frame shear walls in the Northridge earthquake experienced severe damage. GWB is a stiffer material than the structural sheathing, generally oriented strand board (OSB) or plywood. Hence, the greater stiffness of the GWB than the OSB results in uneven load sharing between the two materials and extreme damage to the GWB (Sinha and Gupta 2009). The stiffer GWB attracts the seismic load at low deflections until a brittle failure of the GWB results, and then the load is transferred to the OSB.

McMullin and Merrick (2002) showed that cosmetic damage to the GWB begins at 0.25% drift (lateral wall displacement at the top of the wall) and total economic loss of the wall occurs at 2% drift. Cracking of the GWB panels around the corners of door and window openings was a common failure progression for all the walls.

Sinha and Gupta’s (2009) conclusion that the failure of the GWB occurs before the OSB was witnessed in the aftermath of the 1994 Northridge earthquake (Schierle 2003) and the 1971 San Fernando earthquake (Lagorio 1990). Following the Northridge earthquake, field investigations were carried out by the Consortium of Universities for Research in Earthquake Engineering (CUREE) task force in Los Angeles (Schierle 2003). According to on-site investigations of 1,230 randomly selected buildings, nonstructural (GWB) damage was the most frequent and expensive repair for single family dwellings, costing an average of $7,989 for the building repair (Schierle 2003). Another study on other historic earthquakes found that nonstructural damage can account for a maximum of 70% of future repair costs (Lagorio 1990). For multi-family dwellings, shear walls were the most frequently damaged and expensive to repair, costing an average of $215,765 (Schierle 2003). Failures in nonstructural components and shear walls comprised the majority of the damage to wood frame structures.
in the Northridge earthquake (Seible et al. 1999). A similar study was done on the San Fernando earthquake by Oakeschott (1975). Of the 12,000 buildings surveyed, 89% had some sort of damage to the GWB. Although the majority of this damage could be repaired by simple spackling, the overall cost was still significant because of the high frequency of this problem.

Because of the property loss from the Northridge earthquake related to GWB and shear walls (Schierle 2003), currently implemented shear wall designs should be updated. A major cause of this past damage is from the different stiffnesses of the GWB and OSB (Sinha and Gupta 2009). The stiffer GWB will cause an increased strength and stiffness, but reduced ductility of the wall (Uang and Gatto 2003). CUREE recommends that this change in shear wall behavior be considered in design (Cobeen et al. 2004). Specifically, CUREE recommends future studies to understand how these shear wall components interact with each other. Additional recommendations include that design codes should account for the increased strength from finish materials such as GWB, decreased ductility by reducing the R-factor, and increased initial stiffness at low drifts.

The overall goal of this project was to design a shear wall that can withstand greater displacement before damage occurs to the GWB. This project capitalized on using common materials and practices. Using common practices, these alternative designs can be employed by homeowners and contractors. With common materials, the up front cost of buildings remains similar. Both of these factors allow for an improved design to be quickly and efficiently incorporated into standard practices. By comparing the performance of different wood frame designs with and without window openings, the benefits of these designs could be applied to residential housing. Comparing the behavior of these shear walls required more knowledge of shear wall behavior than was available in the literature. Information regarding visual failure criteria and movement of individual elements of the shear walls was required.

The specific objectives of the project were:

1. To evaluate damage to the GWB in alternative shear wall designs at 1, 2, and 3% drift levels and to compare these results to current performance-based design standards.
2. To quantitatively evaluate the relative displacement between the GWB and the wood frame under monotonic loading.
3. To evaluate the value of alternative shear wall designs considering damage sustained from drift levels.

Methods and Materials

Shear Wall Specimens

All shear walls were 2,440 × 2,440 mm in size and contained double top plates and double end studs. Aside from the headers used in window openings, all framing was 38 × 89 mm (51 × 102 mm or 2 × 4 nominal size) No. 1 and better grade kiln dried Douglas fir (Pseudotsuga menziesii) dimension lumber. Vertical studs for walls were spaced at 610 mm on center (o.c.). Double top and bottom plates, end studs, and all vertical studs in which two stud faces contacted were face nailed together at 610 mm o.c. using two 10d (3.3 × 75 mm) SENCO framing nails. All connections of the top and bottom plates to vertical studs were end nailed with two 16d (3.4 × 88 mm) Bostitch framing nails for each stud connection. Framing nails were all full round head, strip cartridge, smooth shank nails driven using a SENCO SN 65 or Bostitch N90RHN framing nailer.

Walls were vertically sheathed with two 1,220 × 2,440 × 11.1 mm 24/16 APA rated OSB panels. On the opposite side, walls were vertically sheathed with two 1,220 × 2,440 × 12.7 mm GWB panels. OSB panels were connected to the wood frame using 8d (2.9 × 60 mm) SENCO framing nails with two different nail spacings outlined in the individual wall specimen designs. GWB panels were attached vertically to the wood frame using 41.3-mm-long Bugle Coarse Grade 25 drywall screws 305 mm o.c. The edge distance (ED) of all shear wall designs is indicated in their respective figures.

All walls were anchored to a fabricated steel beam that was welded to the strong floor to simulate a rigid foundation. The double end studs were anchored to this fabricated steel beam using two SIMPSON Strong-Tie PHD5-SDS3 and 15.9 mm diameter bolts. Additionally, two 63.5 × 63.5 × 12.7 mm base plates with 15.9 mm diameter anchor bolts were placed 305 mm from the end studs to anchor the bottom plate.

Seven wall specimen designs were used in this project. Of these seven, three had window openings and four did not.

Shear Wall Designs without Openings

IRC Shear Wall Design The International Residential Code (IRC) shear wall design, shown in Fig. 1, was the control design for shear
walls without openings. The IRC shear wall design represented the minimum requirements for a shear wall in the 2009 IRC (ICC 2009), and therefore is called the IRC design.

Separated Stud Shear Wall Design Preliminary testing (Goodall 2010) of the GWB indicated that the distance between where a screw was placed and the edge of a GWB panel affects the strength of the GWB wood frame connection. An increased edge distance was found to result in a stronger connection. The separated stud (SEPSTUD) design, shown in Fig. 2, was built to increase this ED and as a result increase the strength of the GWB connection. To accomplish this, a double bottom plate was installed, as was a double middle stud. The two middle studs were separated by 114 mm to allow for an increased ED (57.2 mm) around the entire GWB panel.

Three Inch Nail Shear Wall Design The three inch nail (3INNAIL) design, shown in Fig. 3, was built to determine how an increase in the strength of the shear wall and stiffness of the OSB side of the wall affects the performance of the GWB. As concluded by Sinha and Gupta (2009), unequal load sharing of the GWB and OSB lead to greater initial damage to the GWB. This unequal load sharing was attributable to the greater stiffness of the GWB than the OSB. Stiffening the OSB side of the wall was thought to cause more equal load sharing between the GWB and the OSB. To increase the stiffness of the OSB side, the OSB to wood frame connection was altered by decreasing the nail spacing by a factor of two. This resulted in an edge nailing of 76.2 mm o.c., or three inches, and indicated by the name 3INNAIL, and a field nailing of 152 mm o.c. A double bottom plate and double middle studs were also added to the wood frame. This was to ensure that the heavy nailing pattern would not split the kiln dried wood studs. The double bottom plate and middle studs also allowed for a larger edge distance for the GWB.

Two OSB Shear Wall Design Like the 3INNAIL design, the two OSB (2OSB) wall design was built to more evenly distribute the load between the GWB and OSB. This design, shown in Fig. 4, was exactly like the IRC design except that OSB panels were
applied to both sides of the wood frame, as indicated by the name 2OSB. The GWB panels were then applied on top of the additional OSB panels. Applying materials with equal stiffness to each side of the wood frame was thought to share load more evenly and put less strain on the GWB. The nailing pattern for the OSB to wood frame connection was the same on both sides of the wood frame as the IRC wall design.

Shear Wall Designs with Openings

IRC Windows Shear Wall Design The IRC windows (IRCWIN) design, shown in Fig. 5, was the control design for shear walls with openings and the adaptation of the IRC design to have window openings. Each shear wall with a window opening had a header and opening sill to frame the opening. The openings were 1.105 × 610 mm, located in the very center of the wall. The header consisted of two 38 × 152 mm (51 × 152 mm or 2 × 6 nominal size) No. 2 and better grade kiln dried Douglas fir \((Pseudotsuga menziesii)\) lumber lengths with an 11.1 mm OSB spacer in between. Jack studs and a cripple stud were placed above the header and beneath the opening sill, respectively. Two 16d nails were used to end nail horizontal members to vertical members such as the header to the vertical studs. The nailing pattern for the OSB to wood frame connection was the same as the IRC design.

2OSB Windows Shear Wall Design The 2OSB windows (2OSBWIN) design, as shown in Fig. 6, was the adaptation of the 2OSB design to have window openings. This design was exactly like the IRCWIN design except that OSB panels were applied to both sides of the wood frame. The GWB was then attached to the top of the additional OSB panels. The nailing pattern for the OSB to wood frame connection was the same on both sides of the wood frame as the IRCWIN design.

Four Panel Windows Shear Wall Design McMullin and Merrick (2002) noted that one method of failure for partition walls with openings was cracking of the GWB near the corners of the openings. The four panel windows (4PNLWIN) design, as shown in Fig. 7, was built to prevent this sort of failure mode. GWB panels

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Fig. 4. 2OSB shear wall design

Fig. 5. IRCWIN shear wall design

Fig. 6. 2OSBWIN shear wall design

Fig. 7. 4PNLWIN shear wall design
were cut and applied such that they connected at the window opening corners. This allowed for a gap between panels near the corners which could allow panel rotation before cracking occurred. Additional jack studs above the windows were required so that the panels could be applied in this way. The nailing pattern for the OSB to wood frame connection was the same as the IRCWIN design.

**Test Setup**

Testing was conducted in the Gene D. Knudson Wood Engineering Laboratory in Richardson Hall at Oregon State University. All walls were anchored to a fabricated steel beam, which was welded to the strong floor to simulate a rigid foundation. The double top plate was bolted to a 2,700-mm-long load head with two 22.2-mm-diameter bolts and two 12.7-mm-diameter bolts spaced from each end at 127 and 330 mm, respectively. The load head was attached to a 49 kN capacity, 254 mm stroke hydraulic actuator. The hydraulic actuator was attached to the reaction wall. This actuator was controlled by an MTS 406 servo controller. A 111 kN load cell was attached to the actuator to provide force measurements while the actuator provided global displacement data (local displacements were measured using other transducers, as described later). Additionally, this actuator was supported by a 102 mm hydraulic cylinder such that the weight of the hydraulic actuator was not applied to the wall. Force and displacement data were recorded on a computer using the LabView 8.6i program. Additional details are given in Goodall (2010).

**Monotonic Testing**

Testing was based on the ASTM E 564-06 (ASTM 2006) standard shear wall static load test. This is a monotonic test and is the
most common test used to evaluate shear behavior (strength and stiffness) of typical wood framed walls. All monotonic tests were stopped at 4.0, 8.0, 12.0, 16.0, 20.0, 24.4, 48.8, and 73.2 mm for failure maps of the walls to be created. The 24.4, 48.8, and 73.2 mm stops were consistent with 1%, 2%, and 3% drift of the wall height. These tests were performed at two different loading rates. For all loading between 0 and 24.4 mm, the loading rate was 4.0 mm/min. For all loading between 24.4 mm to failure, a loading rate of 10.0 mm/min was used.

Visual Failure Comparison

At each stop in the monotonic test, a visual failure map of the wall was constructed. A detailed screw pattern was drawn and numbered on each GWB panel so that all walls with the same design had the same screw attachment and the exact location of each connection failure could be recorded. A visually based failure criteria (described later) was used, which allowed for an objective analysis of screw connection failure. For each failure map, the number of screw connections failed, the failure mode of each connection, and the locations of the failed connections were recorded.

Instrumentation

To determine the relative displacement between the wood studs and the GWB panels in the shear walls, spring return linear position (displacement) sensors were used. These sensors are not a part of the ASTM E64 test setup but were needed to measure the relative displacement, and did not affect any other ASTM required measurements. Sensors were attached in sets of two perpendicular to each other (Fig. 8) and contained a loop attachment at one end of each sensor. One end was bent into a loop such that an 8d nail could be placed through it, and the other end was rigidly attached to the displacement sensor. A hole was drilled through the GWB, and also the OSB for 2OSB and 2OSBWIN designs, and a nail placed through both loop attachments for each set of sensors and into the wood frame (Fig. 8). This allowed the displacement sensors to extend or contract when the GWB panel moved relative to the wood frame. Because these sensors were set up perpendicular to each other, they could measure vertical and horizontal motions.

These panels were attached in four locations, as shown in Fig. 8. These four locations were the lower end stud, upper end stud, lower middle stud, and upper middle stud. The data from panel attachments at all four locations were acquired by having the panels attached at two of the locations in one test, and the other two in another test of an identical wall design. Tests in which the panels were attached to the end (uplift) stud were indicated by an E at the end of the test name. Tests in which the panels were attached to the middle stud were indicated by an M at the end of the test name. These sets of sensors represent the horizontal and vertical relative displacement between the wood frame and the GWB at the locations indicated in Fig. 8.

Test Matrix

A total of 14 walls were tested to failure in this study, as shown in Table 1. Seven shear wall designs were implemented with two walls built per wall design. Each wall design had one wall where the instrumentation panels were attached to the GWB near the end stud, and one wall where the instrumentation panel was attached to the GWB near the middle stud. Six of these walls had window openings, and eight did not. Two additional preliminary wall tests were performed on IRC design walls. The results from these walls were not included in this study, because they were tested to verify the test system, instrumentation, and visual failure modes.

Results and Discussion

Preliminary Testing

The following failure modes of the GWB were observed through preliminary connection testing:

1. Tearing or cracking of the GWB near or around a screw connection [Fig. 9(a)].
2. Minimum of 2 mm of pull-through of the screw into the back of the GWB [Fig. 9(b)].
3. Local crushing of the GWB, indicated by a visible bubble (swelling of GWB) around the screw connection [Fig. 9(c)].

The strength and displacement at maximum load of the GWB screw connections was a function of ED. The average connection strength was similar for 19.1 and 57.2 mm EDs, but approximately 29% lower for the 9.5 mm edge distance than the other EDs. The displacements at maximum load for the 57.2 mm ED were 24 and 19% greater than those observed in the 19.1 and 57.2 mm EDs, respectively. The 57.2 mm ED was utilized in the SEPSW design because of this increased strength and displacement at maximum load. Noticeable visual failures were observed to occur abruptly, not gradually as relative displacement increased. This occurred at approximately 3 mm of displacement, which was taken as the visual failure limit.

Shear Wall Behavior

Table 2 shows the maximum load and stiffness of the shear walls tested. The stiffness of all shear walls was calculated by dividing 80% of maximum load by the global displacement observed at that load. This method was used by Uang and Gatto (2003). ASTM E564 (ASTM 2006) mentions using 33% of maximum load, which results in a much higher stiffness (by a factor of two), but allows the researcher to use other reference load levels to determine the stiffness. Using a lower load level, such as 33% of maximum load, results in an initial stiffness that is more representative of the testing procedure and setup than actual wall behavior. Additionally, using 80% maximum load method was shown to predict cyclic performance from monotonic tests (Uang and Gatto 2003), which is of interest in the application of the results of this study to dynamic earthquake loading.

Shear wall designs 2OSB, 3INNAIL, and 2OSBWIN all exhibited the greatest strengths with similar maximum loads. This similarity can be attributed to the same number of fasteners used to
connect the OSB to the wood frame in all of these designs, which resulted in a stronger connection. Much lower average maximum loads were observed in the IRC, SEPSTUD, IRCWIN, and 4PNLWIN designs owing to fewer fasteners used to connect the OSB to the wood frame, resulting in a weaker OSB connection. Little change in strength was observed between the IRC to IRCWIN design and the 2OSB to 2OSBWIN design, contrary to what was predicted by current design codes. The National Design Specification (NDS) for wood construction predicted a 55% reduction in strength (AFPA 2005), but almost no change was observed.

Much like maximum load performance, shear wall designs 2OSB, 3INNAIL, and 2OSBWIN exhibited the greatest stiffnesses, whereas the IRC, SEPSTUD, IRCWIN, and 4PNLWIN designs exhibited lower stiffnesses. The 2OSB, 3INNAIL, and 2OSBWIN designs all had greater stiffnesses than other designs owing to the greater number of fasteners used to connect the OSB to the wood frame. Because the 4PNLWIN design had four GWB panels attached instead of two, the GWB panels were free to rotate, resulting in the lowest stiffness of all designs.

**Shear Wall Drift Levels and Performance**

Fig. 10 shows the 1, 2, and 3% drift criteria using ASCE/SEI 41-06 (ASCE 2006), the allowable strength using the NDS (AFPA 2005) and the 2% drift limit using ASCE 7-05 (ASCE 2005) for the IRC-E shear wall specimen. The displacement observed at the allowable strength determined by the NDS falls well below any of the drift criteria.

Most buildings in the United States are built to a life safety design level using ASCE 7-05 (ASCE 2005), which sets a 2% drift limit on most wood frame structures. This is consistent with the ASCE/SEI 41-06 (ASCE 2006) life safety structural performance level. According to the ASCE/SEI 41-06 life safety structural performance level, “...a structure has damaged components but retains a margin against onset of partial or total collapse...” In wood stud walls, this occurs at 2% transient drift and relates to “moderate loosening of connections and minor splitting of members.” Shear walls were observed to have minor member splitting and connection loosening at 2% drift, which is consistent with the ASCE 41-06 life safety performance level description. Performance at other drift levels is discussed in Goodall (2010).

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<th>Table 1. Test Matrix</th>
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**Fig. 9.** GWB failure modes: (a) tear or crack; (b) pull through; (c) crushing-bubble
Shear Wall Visual Failure Comparison

Shear wall loading was stopped at 10 different displacements, including 1, 2, and 3% drifts, where visual failure maps were created. A percentage of the global connection failures were computed at each displacement stop for each wall. This percent global connection failure was the number of failed fastener connections divided by the total number of fasteners used to connect the GWB panels to the wood frame for a particular wall design.

Shear Wall Comparison at Drift Criteria

A visual connection failure comparison between shear walls is shown in Fig. 11. At 1% drift, all innovative shear wall designs outperformed the control designs. The SEPSTUD and 4PNLWIN designs performed moderately better than the IRC and IRCWIN designs, but the 3INNAIL, 2OSB, and 2OSBWIN designs exhibited the best GWB performance.

At 2% drift, the SEPSTUD and 4PNLWIN design performed worse than the IRC and IRCWIN designs, although the 3INNAIL design performed only slightly better. The 2OSB and 2OSBWIN designs exhibited superior GWB performance with only 11 and 7% global connection failure, respectively. The IRC and IRCWIN designs exhibited 62 and 65% global connection failure, respectively. These large global connection failures in the IRC and IRCWIN designs are consistent with the large economic loss observed in previous test results of GWB partition walls (McMullin and Merrick 2002), which identified a total economic loss of the GWB at 2% drift.

Similar trends were shown at 3% drift, with the 4PNLWIN and SEPSTUD designs exhibiting similar performance to the IRC and IRCWIN designs. The 3INNAIL design performed only slightly better than the IRC design at 3% drift. However, the 2OSB and 2OSBWIN designs exhibited superior GWB performance even at 3% drift. Although all innovative designs performed better at 1% drift, the 3INNAIL, 2OSB, and 2OSBWIN designs performed better at all drifts, with the 2OSB and 2OSBWIN designs exhibiting superior overall performance.

In all wall designs aside from the 2OSB and 2OSBWIN designs, a minimum of 51% global connection failure was observed at 2% drift. This implies that walls loaded to 2% drift, or the maximum ASCE 7-05 allowable displacement (ASCE 2005), will sustain substantial damage to the GWB in a seismic event. This amount of damage is consistent with what was observed in the aftermath of the Northridge earthquake (Seible et al. 1999).

The 3INNAIL, 2OSB, and 2OSBWIN designs all had the highest strengths and exhibited the best GWB performance.
As expected, increasing the strength and stiffness of a shear wall made it more resistant to damage in the GWB.

The 2OSB and 2OSBWIN designs performed so well owing to similar stiffness of both sides of the shear wall. In all other walls, the stiffer GWB on one side of the wall attracted more load until the GWB began to transfer load to the OSB (Sinha and Gupta 2009). If OSB was applied to both sides of the frame, and GWB attached to the OSB, both sides of the frame would have similar stiffness. This similar stiffness allowed the OSB on both sides of the frame to share load equally and apply less load to the GWB. With less load applied to the GWB, less damage would occur to the GWB.

Shear Wall Relative Displacement Comparison

Qualitative Relative Displacement Comparison
The relative displacement of the GWB to the wood frame was quantified by the measurements made by the displacement sensors. The displacement vectors of the IRC design shear wall at 1% drift are shown in Fig. 12. The displacement vectors for all other walls and associated discussion are given in Goodall (2010). One percent drift was chosen because it was the highest level of displacement at which the sensors could be displaced where a visual failure map was recorded. The displacement vectors in Fig. 12 are shown at their true angles and scaled 100 times their length. The directions of the relative displacement vectors are consistent with those predicted by the classical GWB rotation model (Cobeen et al. 2004; Foz and Filiatrault 2001).

Displacement Vector Magnitude Comparison
In the preliminary GWB connection testing (Goodall 2010), visual failures occurred abruptly, not gradually, with relative displacement. The average displacement at which visual failures were observed in preliminary testing was 3 mm. This was taken as the average visual failure limit, or the relative displacement at which visual failure criteria was met.

The magnitudes of the displacement vectors for shear walls without openings at 1% drift are shown in Fig. 13. The 40% global connection failure observed in the IRC wall at 1% drift, shown in Fig. 11, is consistent with the six relative displacement vector magnitudes above the visual failure limit. This implies that much of the GWB panel was displaced above the 3 mm visual failure limit.

The 31% global connection failure observed in the SEPSSTD design at 1% drift, shown in Fig. 11, is consistent with two relative displacement vector magnitudes above the visual failure limit. The SEPSSTD design had a 33% larger top middle plate displacement vector magnitude than the 3INNAIL design. This is because of the greater uplift of the top plate, because there was no connection between the two middle studs in the SEPSSTD design. This top plate vector was the largest recorded relative displacement magnitude and explains the large global connection failure in the SEPSSTD design at 1% drift. Similar to the 3INNAIL design, visual failures were recorded near the top middle plate and bottom end plate. Only two visual failures were recorded near the middle stud of all the 3INNAIL or SEPSSTD design walls. This is much less than for IRC design walls, where nearly all fastener connections on the middle stud were determined to have failed at 1% drift. This implied that in the double middle stud walls, the GWB panel mostly stayed attached to the middle stud, but not to the top plate.

The 3INNAIL design exhibited an 8% global connection failure at 1% drift, which was concentrated near the top end plate and bottom end plate. This is consistent with vector magnitudes greater than the visual failure limit of the bottom end plate and top middle plate. These vectors are close in magnitude owing to the heavy sheathing attachment and attachment of the middle studs, which resulted in a greater force transfer.

At 1% drift, the 2OSB design exhibited a 3% global connection failure, as shown in Fig. 11. The resultant few fastener connection failures are consistent with relative displacement vector magnitudes below the visual failure limit. The bottom end stud vector is very close to this limit with a magnitude of 2.98 mm, but the 2OSB design still exhibited the least amount of relative displacement above the visual failure limit than all other designs.

The magnitudes of the displacement vectors for shear walls with openings at 1% drift are shown in Fig. 14. Similar to the IRC design, the IRCWIN design exhibited a 38% average global connection failure at 1% drift, as shown in Fig. 11. This is consistent with the six displacement vector magnitudes above the visual failure limit. Four of these vectors are the same as for the IRC design.

Both 2OSBWIN and 4PNLWIN designs exhibited no relative displacement magnitudes above the visual failure limit. Although this is consistent for the 2OSBWIN design, in which only a 1% global connection failure was observed, it is not consistent for the 4PNLWIN design, in which 17% global connection failure was observed. Because of the very different GWB panel attachments in the 4PNLWIN design than the other designs, the relative displacement vector magnitudes may not be comparable to other designs. Additionally, the relative displacement magnitudes observed at the middle stud in the 4PNLWIN design are less than those of other walls because they are measured near the middle of the GWB panel, not the edges. The GWB panel rotates about its center, which causes more displacement near the edges of the panel than near the center.

Except in the 4PNLWIN design, shear wall designs with more relative displacement measurements above the visual failure limit exhibited more visual GWB damage, which validates the visual failure results. However, this type of measurement may not be the optimum way to determine damage to GWB. For example, the SEPSSTD design exhibited much higher visual damage than 3INNAIL, even though it exhibited the same number of relative
displacement vector magnitudes above the visual failure limit. Perhaps more sensor measurements along the panel edge between studs or at intermediate studs would give a more complete picture of the relative displacements of the GWB panels in all instances, and give a better indication of visual failure behavior.

Shear Wall Value Comparison

Implementation of any type of new construction method is directly related to the cost and value of using that method. Excluding labor cost and based on material cost only, a 27% increase in material cost was observed for the IRC to 2OSB and IRCWIN to 2OSBWIN designs. This increase in cost must be justified by an increase in performance for the 2OSB and 2OSBWIN designs to be implemented as a new construction method.

To investigate how each shear wall performed in relation to its material cost, a shear wall value comparison was performed. The goal of this value comparison was to rank the performance of each shear wall design while considering their material cost and to see which shear wall design demonstrated the most efficient use of materials. The shear wall value index was calculated by dividing the percent global remaining connections at 1 and 2% drift levels by the material cost of the shear wall. Three percent drift level was not included, because the cost of the GWB would not be an important factor in repairs for a building subjected to 3% drift. The percent global connection failure was calculated from the visual failure results and the percent global remaining fasteners was calculated by 100% global connection failure.

This value comparison for all shear wall designs is shown in Fig. 15. All innovative shear wall designs exhibited greater value than the control at 1% drift. The design with the greatest value was the 3INNAIL design, with the largest shear wall value index compared to the IRC design. This shows that the 3INNAIL design demonstrates the most efficient use of materials at 1% drift. The 2OSB and 2OSBWIN designs also displayed higher value than the IRC or IRCWIN designs at 1% drift.

At 2% drift, the range in shear wall value indexes was much higher. The 2OSB and 2OSBWIN designs exhibited the greatest value, with the greatest increases in shear wall value index from the IRC or IRCWIN designs. These results showed that sheathing a shear wall with OSB on both sides resulted in the most efficient use of materials at 2% drift.
Although the 3INNAIL design exhibited the greatest value at 1% drift, the 2OSB and 2OSBWIN designs demonstrated a greater value overall. This is because shear walls are generally built to be displaced to the life safety drift (ASCE 2005; ASCE 2006) of 2%. At this 2% drift, the 2OSB and 2OSBWIN designs both showed superior value above all other shear walls.

This shear wall value index can only be used for comparison purposes. It is difficult to assign an economic value to each connection failure, and the economic losses to shear walls are more likely a step function than a continuous loss (McMullin and Merrick 2002). Additionally, only the material cost of each shear wall was considered. In construction, the added labor cost for two additional OSB panels on the wood frame was much greater than adding additional nailing or a middle stud. By not including this labor cost, the value comparison favored the 2OSB and 2OSBWIN designs. To determine the additional labor costs, a professional framer would need to be hired to build the walls.

Conclusions

Conclusions based on the results of this study include:
1. ASCE/SEI 41-06 drift criteria generally agreed with observed behavior of all shear walls.
2. Aside from the 2OSB and 2OSBWIN designs, substantial damage to the GWB was observed in wall designs at the ASCE 7-05 drift limit of 2% drift with a minimum of 51% global connection failure.
3. Less GWB damage was observed in the 3INNAIL, 2OSB, and 2OSBWIN designs than in the IRC or IRCWIN designs at all drifts. Increasing the stiffness and strength of a shear wall resulted in less GWB damage for a given loading or displacement.
4. Adding OSB to both sides of a shear wall (2OSB and 2OSBWIN designs) resulted in superior GWB performance up to failure. This is because the similar stiffness on both sides of the wood frame resulted in equal load sharing and less damage to the GWB.
5. Increasing the strength of the GWB connection by using a larger edge distance (SEPSTUD design) improved GWB performance up to 1% drift, but affected the performance negatively at 2 and 3% drifts.
6. Applying panels around window openings (4PNLWIN design) increased GWB performance up to 1% drift, but affected the performance negatively at 2 and 3% drifts.
7. Little damage was observed in the GWB for walls loaded to the allowable strength. This implied that walls damaged in the Northridge earthquake were loaded above allowable strengths. The shear walls were more likely loaded beyond 1% drift.
8. The visual failure limit exhibited in small GWB samples, observed to be 3 mm in this study, was consistent with behavior observed in shear wall specimens. Shear walls with larger relative displacement above this visual failure limit exhibited more visual GWB damage.
9. The 3INNAIL, 2OSB, and 2OSBWIN designs exhibited more efficient use of shear wall materials at 1 and 2% drifts. The 3INNAIL design exhibited the most efficient use of shear wall materials at 1% drift and the 2OSB and 2OSBWIN designs exhibited the most at 2% drift. Because most buildings are built to be displaced to 2% drift in an earthquake, sheathing shear walls with OSB on both sides is the most efficient use of materials.
10. The 3INNAIL, 2OSB, and 2OSBWIN designs exhibited more efficient use of shear wall materials at 1 and 2% drifts. The 3INNAIL design exhibited the most efficient use of shear wall materials at 1% drift and the 2OSB and 2OSBWIN designs exhibited the most at 2% drift. Because most buildings are built to be displaced to 2% drift in an earthquake, sheathing shear walls with OSB on both sides is the most efficient use of materials.

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