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Postpeak residual capacity of nailed connections of a shear wall

Abstract: To quantify the postpeak residual capacity and to gain insights into the load transfer mechanism of a shear wall, nail connection tests were performed on salvaged connections after a monotonic shear wall test loaded up to peak load. Experimental results reveal that there is a loss of strength in most of the fasteners studied, indicating that almost all the fasteners contribute toward racking resistance of the wall. The maximum loss of strength was observed for the fastener in the uplift corner and for the fastener along the middle stud. Another area where fasteners exhibited a significant loss of strength was in plate connection located at the bottom plate. The performance of a shear wall can be enhanced by strengthening the two areas – uplift corner and bottom plate.

Keywords: bottom plate, load transfer, nail connection test, national design specification, racking resistance, strength loss, uplift corner, wood-frame shear wall, yield strength

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Introduction

A majority of the residential structures in the US are wood-frame construction. Historically, wood-frame structures have performed very well under seismic loading. However, the damage assessment after the 1994 Northridge Earthquake raised a few concerns about the performance of wood-frame construction (Cobeen et al. 2004). Post the Northridge earthquake, a series of research projects were instigated to quantify the performance metric of wood-frame walls (Cobeen et al. 2004) and also to look into the load-sharing aspect of the sheathing materials used (Sinha and Gupta 2009).

A competent lateral force resisting system (LFRS) is highly dependent on a shear wall. In a wood-frame construction, shear walls are constructed using a dimension lumber such as 2×4 (38×89 mm) framing members. An oriented strand board (OSB) or plywood is also applied as a structural sheathing connected to the framing by dowel-type fastener, such as nails. On the other side, a gypsum wall board (GWB) is used as a finish material attached to the framing by regular drywall screws. A shear wall dissipates the seismic energy through the yielding of sheathing to framing connections. Shear walls have been the subject matter of much research, which is well reviewed and documented (van de Lindt 2004; Kirkham et al. 2013).

The lateral load-carrying capacity of a shear wall is a function of connection stiffness, the shear stiffness of the panel, and the bending stiffness of the framing material, with connection stiffness being the predominant factor. Similar to shear wall, sheathing to framing connections has been widely studied experimentally (Foschi 1974; McLain 1975; Foschi and Bonac 1977; Price and Gromala 1980; Aune and Patton-Mallory 1986a; Kent et al. 2004; Anderson et al. 2007; Sinha et al. 2011; Li et al. 2012) as well as numerically (Kuenzi 1955; Aune and Patton-Mallory 1986b; Smith et al. 2001; Nishiyama and Ando 2003). There is, however, a lack of studies on quantifying the residual strength of the connections once the ultimate capacity of a shear walls is reached. This information can serve two purposes. First, this information will provide detailed insights on the load transfer mechanism in typical shear walls. Second, this information is needed to assess the postpeak behavior of the wall, which in turn can help quantify the postevent residual capacity of the wall. This information is vital for designing the rehabilitation and retrofit plans. The objective of this study is to find the residual strength of the various OSBs to stud the connection in a shear wall after it has reached the ultimate wall capacity to gain an insight into the load transfer mechanism of a shear wall.

Materials and methods

Wall specimens

A (one) shear wall test specimen was designed and constructed in accordance with the 2000 International Residential Code-prescribed braced panel construction. The wall was 2440×2440 mm in dimension and constructed by means of stud-grade 38×89 mm kiln-dried

Douglas-fir framing as shown in Figure 1a. The framing studs were spaced at 610 mm on the center, connected to the sill plate and the first top plate with two 16d (3.33×82.6 mm) nails per connection, driven through the plates and into the end grain of the stud. A second top plate was connected to the first top plate with 16d nails at 610 mm on the center. The walls were sheathed with two 1220×2440×11.1 mm OSB panels that were attached vertically to the wall frame. The 24/16 APA-rated OSB panels (Ainsworth, Vancouver, BC, Canada) were connected to the wall frame with 8d (2.87×63.5 mm) ring shank sheathing nails (Sheather Plus, Stanley, East Greenwich, RI, USA) spaced 102 mm on the center along the panel edges and 305 mm along the intermediate studs (field nailing). The walls were additionally sheathed with two 1220×2440×12.7 mm GWB (Pabco Gypsum, Newark, CA, USA) panels installed vertically on the face opposite to the OSB structural panels. The GWB panels were attached to the framing with bugle head coarse wallboard screws (2.31×41.3 mm) spaced 305 mm on the center along the panel edges and intermediate studs. The sheathing to framing connections was staggered (not shown in the figure) on the end post and top plate. The double-end studs were required because the walls were anchored with hold-downs and were connected together with 16d (3.33×82.6 mm) framing nails at 305 mm on the center. The framing nails were full round head, strip cartridge, and smooth shank SENCO® (Cincinnati, OH, USA) nails that were pneumatically driven with a SENCO® SN 65 nail gun. The sheathing nails were Stanley Sheather Plus nails driven pneumatically as well.

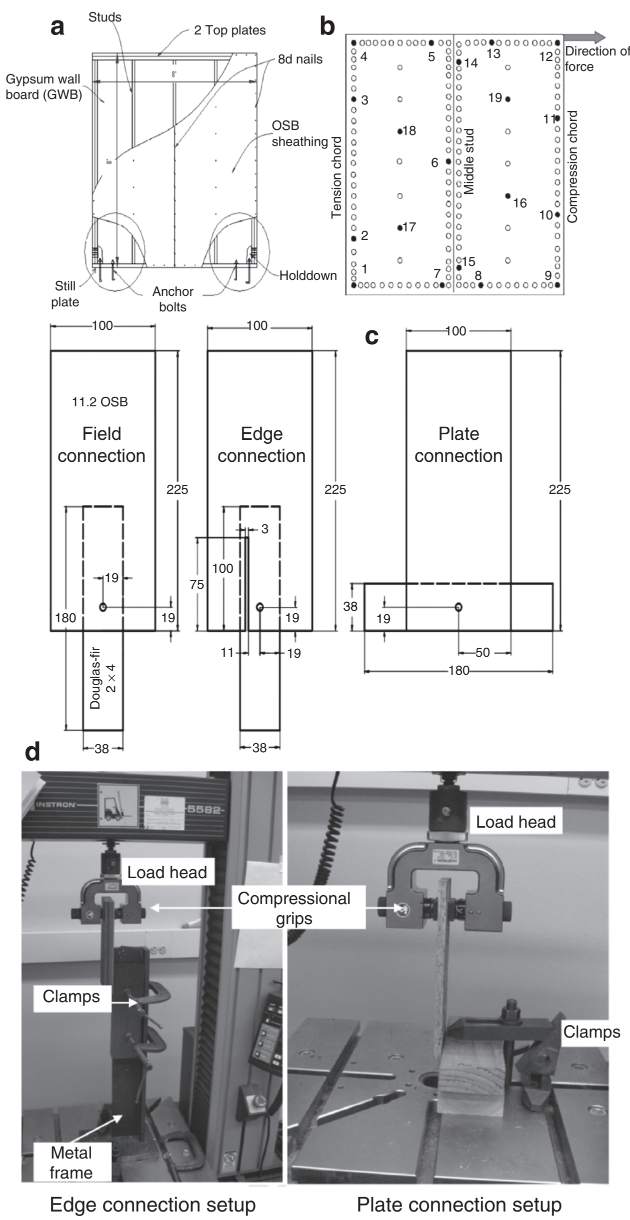

Figure 1 Schematics and photography of some experimental details: (a) shear wall, (b) shear wall cut diagram and fastener numbering, (c) control specimens, and (d) nail connection test set up.

Once constructed, the shear wall was bolted to a fabricated steel beam firmly attached to the strong floor to simulate a fixed foundation. The wall was loaded by means of a 490 kN servo-controlled hydraulic actuator with a 250 mm total stroke. The hydraulic actuator was attached to the strong wall and supported by a 102 mm hydraulic cylinder. A 111.2 kN load cell attached to the piston provided force measurements. A steel C-channel, laterally braced to the strong wall, was attached to the load cell and hydraulic actuator. The C-channel was connected to the top plate of the wall using four evenly spaced 12.7 mm A307 bolts installed through both top plate members. For ensuring a tight nonslip bolted connection, 12.7 mm holes were drilled in the top plates after the walls were positioned. The shear wall was tested monotonically based on the ASTM E564-06 (ASTM 2006) test protocol at a recommended loading rate of 0.76 mm min^{-1}. The test was stopped after the peak load was attained and the load deflection diagram was coming downward but before the failure of the wall. The wall was then cut into various samples for single fastener tests as per the cut diagram illustrated in Figure 1b.

Nail connections

Once the shear wall was tested, a total of 19 connections were salvaged from the wall. The cut diagram (Figure 1b) shows the connections from three distinct areas of the wall: (1) edge specimen (ES), which is located at the tension, compression, and middle chord of the wall (connections 1–4, 6, 9–12, and 14–15); (2) field specimen (FS), which is located at the intermediate studs (connections 16–19); and (3) plate specimen (PS), which is located at top or bottom plate of the wall (connections 5, 7, 8, and 13). The edge and field connections are similar, the only difference being the distance between the edge of the panel and the nail is less in ES than in FS. The PS differs from the other two in the direction of loading.

The three types of control specimens representing the edge, field, and plate nailing conditions are shown in Figure 1c. The nails were laterally loaded by means of a Universal Testing Machine (UTM, Instron 5582) at a constant displacement rate of 5 mm min^{-1}. Two setups were used to hold the loading geometries, namely, edge and plate, as depicted in Figure 1d. The connections were designed by modifying the ASTM D 1761 (ASTM 2007). Compression clamps gripped the sheathing on top for both geometries. The framing was clamped to a right angle metal bracket for the ES and FS, while for the PS the framing was clamped to the floor of the UTM as visible in Figure 1d. This apparatus kept the specimen straight and in-plane to reduce the eccentricities caused by nail withdrawal. The slip surface was specifically centered on the centerline of the load head to reduce the eccentricity. Furthermore, especially in plate connection geometry (Figure 1d, right), the clamps holding the wood down to the base plate are attached in such a way that it has minimal influence on the shear deformation of the fastener. The load-slip (P-D) curves were recorded for each test. The test was stopped after a plateau had been reached in the P-D curve. The National Design Specification (NDS) yield limit load refers to the 5% diameter offset load, defined as the intersection of the P-D curve and a line parallel to the initial linear portion of the P-D curve offset by 0.05 times the shank diameter of the dowel in the positive direction (AFPA 2012). The yield strength was calculated from the P-D curves by the 5% diameter offset method. Here, the 5% offset was 0.14 mm. The ultimate load from the P-D curve is the maximum load the connection can withstand without failure. However, the yield models in NDS suggest that, for a connection, the yield strength is considered to be the ultimate strength for the connection (Aune and Patton-Mallory 1986b; Sinha et al. 2011; AFPA 2012). Moreover, the design of connections is based on this assumption. Hence, for the present study, the yield strength was considered as the ultimate strength.

Results and discussion

The strength data of all connections salvaged from the shear wall as well as of the controlled samples are listed in Table 1. The load deflection curve for the shear wall loaded to the ultimate load and then stopped is presented in Figure 2a. The average lateral strength of connection is 1080 N for edge nail and 898 N for field nail; for plate geometry, the average strength is 588 N (Table 2). The average load data are presented in Table 2 for the controls and salvaged connections grouped according to the connection geometry.

Edge connections

A total of four fasteners were tested from the tension chord. The maximum loss of strength in tension chord was at the fastener in the uplift corner (corner between tension chord and the sill plate) at the sill level (fastener 1). Its residual strength was 528 N, which is about 49% of the control connection yield capacity. As moving up along the tension chord toward the top plate, the magnitude of loss of strength decreases; hence, the residual strength increases. Fastener 4, which is at the top of the tension chord, has utilized only 30% of its strength in the shear wall test.

Like the tension corner, the fastener analyzed at the compression corner also had a residual strength of 49% of the actual fastener capacity, while the other fasteners at the compression chord had a residual strength of 74% and 68% of the ultimate nail capacity. Fastener 12, which is at the junction of the compression corner and the top plate, had 67% of the connection strength unutilized during the shear wall test.

Nails 17 and 18 are on the intermediate stud near the tension chord, while nails 16 and 19 are on the intermediate stud near the compression chord of the wall. Nail 17 achieved a peak load of 1014 N and a yield load of 461 N, which is half the average yield capacity of the field nail geometry controls. Nail 19 attained a peak load as well as calculated yield load, which is slightly greater than the average value obtained from the control specimen. This

Table 1 Results from shear wall connection tests and controls.

Nail no.	Location	Geometry	Residual strength (N)	Average control loads (N)	Loss in strength (N)	Loss in strength (%)
1	Uplift corner	Edge	528	1080	552	51
2	Tension chord	Edge	576	1080	504	47
3	Tension chord	Edge	655	1080	425	39
4	Tension chord and top plate	Edge	758	1080	322	30
5	Top plate	Plate	310	578	268	46
6	Middle stud	Edge	785	1080	295	27
7	Bottom plate	Plate	368	578	210	36
8	Bottom plate	Plate	233	578	346	60
9	Compression corner	Edge	530	1080	550	51
10	Compression chord	Edge	801	1080	279	26
11	Compression chord	Edge	738	1080	342	32
12	Compression chord and top plate	Edge	726	1080	354	33
13	Top plate	Plate	414	578	164	28
14	Middle stud	Edge	635	1080	445	41
15	Middle stud	Edge	372	1080	708	66
16	Intermediate stud	Field	791	898	107	12
17	Intermediate stud	Field	461	898	437	49
18	Intermediate stud	Field	641	898	257	29
19	Intermediate stud	Field	904	898	-6	0

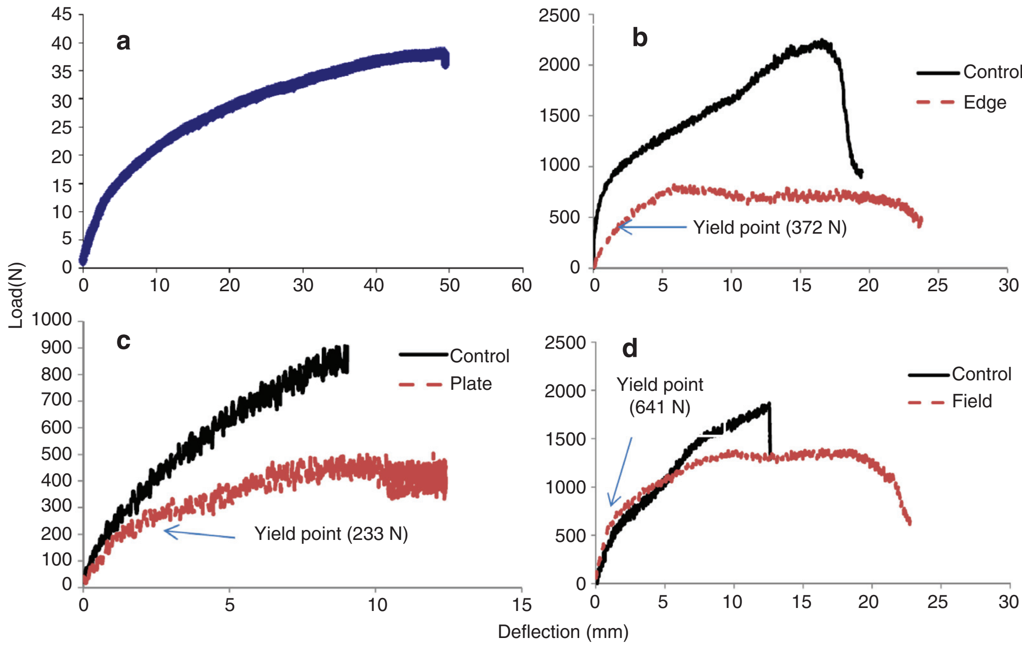

Figure 2 Load deflection diagram for (a) shear wall racking test until ultimate load; load deflection diagram for control and a typical connection for (b) edge specification (nail 15), (c) plate specification (nail 8), and (d) field specification (nail 18).

could be an artifact of the variability associated with wood such that the load capacity of connection was higher than the controls. Another reason could be that this nail does not contribute to the racking resistance of the wall. The higher capacity of nail could also be attributed to the difference in the stiffness values of the underlying stud in which the nails are driven. The residual capacity of nail 16 was 88% of the ultimate connection capacity and that for nail 18 was 71%. The nails (16 and 19) in the intermediate stud that are close to the compression chord are less utilized during the monotonic wall test than the ones near the tension corner, as these nails have much higher residual capacity.

Plate connections

Two nails were tested at the bottom plate level from either panel. Nails 7 and 8 had a residual strength 64% and 40% of the ultimate capacity, respectively. Nail 5, which was located at the top plate, attained a peak load of 698 N. This corresponds to 310 N yield load, which is 54% of the yield load capacity, while nail 13 on the other panel had a capacity of 443 N, which is 72% of the yield capacity of the fastener under plate geometry. Nail 5 is very close to the middle stud and nail 13 is more toward the middle of the panel. The close proximity to the middle stud, where most of the failure is, makes its residual strength much lower compared to nail 13.

Table 2 Summary of test results classified as per connection geometry.

Geometry	Controls		Salvaged connections	
	Average load (N)	SD (N)	Average load (N)	SD (N)
Edge	1080	193	646	127
Plate	578	46	699	191
Field	898	80	331	78

Field connections

Three nails were tested from the middle stud of the wall. One nail (15) was from near the sill plate, one was from the central portion (nail 6) of the wall, and the last one (nail 14) was located nearer to the top plate. The residual capacity of nail 14 was 59% of the fastener capacity, while for nails 6 and 15 it was 73% and 34%, respectively.

General discussion

Figure 2b–d permits the comparisons of load deflection between the control and a typical nail of that geometry. For the edge geometry (Figure 2b), the nails cut from the walls show a loss of strength and elastic stiffness. Both the connection stiffness and strength were severely impacted due to the initial loading of shear wall. Nail 15 suffered the highest loss of strength, as its residual capacity was reduced to 34% of the actual fastener capacity. This is expected because, along with nail 1, nail 15 represents the critical fastener position essential for a proper shear transfer. Similar to the field nails, a loss of strength and stiffness is observed. Field nails, especially those that are located at the upper half of the wall, do not play a proactive role in the load transfer in the structural sheathing of the shear wall (Sinha and Gupta 2009); hence, a negligible strength loss is observed during the racking test. The stiffness of field nail is almost the same as the connection controls.

Table 1 shows that the strength loss of majority of the connections after the wall has been subjected to ultimate load (Figure 2a) is <50%. This observation is testimonial to the weakest link model for a shear wall, which states that the wall is as good as its weakest link. As seen from these test, fasteners near the bottom of the wall had much less residual capacity than other fasteners considered in this study. Therefore, the bottom fasteners are the ones that are governing the ultimate strength of the wall. Furthermore, the fastener in the middle of the wall and near the bottom plate (nail 15) has the lowest residual strength of all fasteners considered in the study. This is because the OSB of the shear wall with GWB generally fails in the uplift corner and along the middle stud of the wall. Moreover, the failure is more in sill plate than in top plate (Sinha and Gupta 2009). The nails at the tension and compression corner have the lowest residual strength, if the compression and tension chords are only considered. The residual strength increases along those chords. The residual strength of nails in tension chord is less than that of compression chord; this is because for the walls the uplift corner is of prime importance and most of the initial activity is concentrated in the tension corner. It is evident that the bottom half of the wall contributes more to the load transfer mechanism of shear through the wall than the top half of the wall. Therefore, nails near the bottom of the wall are experiencing more stress compared to other nails during racking of the wall, which accounts for their low residual strength. Most of the fasteners considered in this study showed some loss of strength during the shear wall racking test and some more than the others; hence, all the fasteners are involved in the load transfer in the shear wall to varying extent depending on their positions. Nails along the middle stud are the ones showing the highest loss, and this is due to the inadequate edge distance for the nails on the middle stud. Over the middle stud, which comprises a 2×4, two OSB panels meet and the connections are driven into the single stud, which gives rise to inadequate edge distance for the connection to develop desired strength and stiffness. This geometry with inadequate edge distance is hard to achieve in the controls; hence, this study had to be content with the standard control for edge geometry. For a more realistic assessment, special control samples are needed to be designed for the single middle stud situation.

Due to the different geometry of the connection, the control for plate connection and edge connection has a lot of variation. From Figure 2, it is observed that the strength of plate geometry is about 60% of the edge geometry and also there is a large variation in the elastic stiffness of the two types of connection geometry. This is in concert with previous studies (Kent et al. 2004; Sinha et al. 2011). Hence, along with the middle stud, due to the inadequate edge distance, the bottom plate also can be considered as a weak link in the shear wall assembly due to the connection geometry (Dolan 1989). That is the reason behind observing the majority of damages at sill level and middle stud (Dolan 1989; Sinha and Gupta 2009). The performance of a shear wall can be enhanced by strengthening these two areas. A double stud can be used in the middle of the wall to eliminate the lack of adequate edge distance. For plate, more research is needed to draw more generalized conclusion; however, some different type of connection or different nailing pattern might provide better results. Caution must be taken when generalizing these results, as this study is based on fasteners salvaged out of one wall. A similar study on multiple walls tested with different loading protocol should be considered in the future.

Conclusions

There is a loss of strength in most of the fasteners studied, indicating that almost all the fasteners contribute toward racking resistance of the wall. The loss of strength of the connections is greater for the connections in the bottom plate and the middle stud region. Most of the damage in racking of shear wall is observed in those areas and hence a higher loss of strength of the fasteners. Field nails, especially in the top half of the wall, has a very low loss of strength, indicating that it is not a major contributor to the racking resistance of the wall. In general, fasteners near the bottom plate have less residual strength than the fasteners in the top half of the wall.

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