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IMPACT OF GRAVITY LOADS ON THE LATERAL PERFORMANCE OF LIGHT GAUGE STEEL FRAME / WOOD PANEL SHEAR WALLS

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ABSTRACT

Methods for the design of steel frame / wood panel shear walls used as a seismic force resisting system have been developed. These methods, which can be used in conjunction with the 2005 NBCC, were based on the results of shear wall tests carried out using lateral loads alone. The research program was extended to determine the impact of gravity loads on shear wall behaviour. A series of physical tests were performed on five single-storey shear wall configurations. The walls were tested under monotonic and cyclic lateral loading, where two of three shear walls were also subjected to a constant gravity load. In total, 32 steel frame / wood panel shear walls composed of 1.09 – 1.37 mm thick steel studs sheathed with DFP, CSP or OSB panels were tested and analyzed. The equivalent energy elastic-plastic analysis approach was used to determine design values for stiffness, strength, ductility and overstrength. The data from this most recent series of tests indicates that the additional gravity loads do not have a detrimental influence on the lateral behaviour of a steel frame / wood panel shear wall if the chord studs are designed to carry the combined lateral and gravity forces following a capacity based approach. A resistance factor of 0.7 was found to be in agreement with previous tests that did not include gravity loads. The calculated seismic force modification factors also agreed with the previous test results, which suggest that R_d = 2.5 and R_o = 1.7.

Introduction

At present, there exists no code or standard in Canada that contains information on the seismic design of lateral systems constructed of cold-formed steel members. In light of this, a study of light gauge steel frame / wood panel shear walls has been ongoing at McGill University since 2000. The general objective of the study is to develop a Canadian design standard that can be used in conjunction with the National Building Code of Canada (NBCC) (NRCC, 2005). To-date, over 150 tests have been completed on shear walls subjected to lateral in-plane loads. A design method has been developed by Branston et al. (2006a,b) based on the analysis of the shear wall test data using the equivalent energy elastic-plastic approach (EEEP) (Park, 1989; ASTM E2126, 2005; Branston, 2004). Analytical models to calculate the resistance and lateral deflection for various configurations of walls using the strength and stiffness characteristics of the sheathing connections have been evaluated (Chen, 2004; Chen et al., 2006). Boudreault (2005) recommended preliminary seismic force modification values, R_d = 2.5 and R_o = 1.7,

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using the measured ductility and overstrength of the test walls. Blais (2006) advanced this work by performing additional shear wall tests, as well as non-linear time history dynamic analyses for two representative buildings under ten earthquake ground motion records from the west coast of North America scaled to the 2005 NBCC uniform hazard spectrum for Vancouver BC (Boudreault et al., 2007). Most recently, Rokas (2006) has recommended design values for cold-formed steel walls sheathed with 9.5 mm plywood sheathing.

The shear wall research at McGill University that had been completed prior to this study by Hikita (2006) consisted of the testing of walls that were subjected to lateral loads alone. A concern existed regarding the inelastic lateral load carrying behaviour of cold-formed steel shear walls when they must also carry gravity loads. In studies of laterally loaded shear walls performed by Serrette et al. (1996), Morgan et al. (2002) and Branston (2004) it was shown that test walls with dense framing to sheathing fastener schedules (75 mm or less) along their perimeter may exhibit local buckling of the chord studs. Since in a real structure, these chord members generally carry compression forces due to the applied lateral loads in combination with gravity loads, the possibility of this failure mode and the effect it would have on the recommended design values provided by Branston et al. (2006a) and Boudreault et al. (2007), among others, needed to be investigated. Hence, a study of the behaviour of light gauge steel frame / wood panel shear walls under combined gravity and lateral loading was completed by Hikita (2006). The objective of this particular study was to determine whether the presence of gravity loads affected the lateral inelastic load carrying performance of cold-formed steel shear walls. The scope of the research included a review of the capacity based approach used for the design of seismic force resisting systems (SFRS), tests to evaluate the axial compression load carrying capacity of the sheathed chord stud members, and the physical testing of shear walls under combined gravity and lateral loading.

This paper describes the results of the shear wall tests. The wall performance was evaluated in terms of shear strength, ductility, overstrength, seismic force modification factors, resistance factor and stiffness. A capacity based design approach was implemented in the selection of all components for the shear wall specimens. An assumption was made, based on the previous tests carried out at McGill University, that the sheathing connections would provide the best means of dissipating energy in the SFRS. The same strategy is applied to the design of traditional wood framed shear walls. The chord studs in a steel framed wall may be prone to failing in compression; hence, the presence of gravity loads may change the failure location from the sheathing connection to the chord studs. Since the chord stud mode of failure is not desirable, because the overall strength and ductility of the shear wall would likely be reduced, the perimeter studs for the test walls were selected such that they were able to carry the applied compression force originating from the combined lateral and gravity loads.

Shear Wall Test Program

This test program consisted of five different configuration shear walls, which were constructed of a cold-formed steel stud frame and a single sheet of plywood or OSB (Table 1). Each specimen measured 1220 x 2440 mm, but varied in chord stud thickness, sheathing type and fastener schedule. The configurations were chosen based on previous tests for which chord failure was of concern, and such that they would be applicable to a variety of designs. In particular, the DFP sheathed panel with 75 mm screw spacing along the perimeter was included in the study because a wall specimen of this configuration, test 13B (monotonic) by Branston (2004), failed due to the local buckling of the chord stud when lateral loads alone were applied. Three walls per configuration were tested under monotonic loading and three were tested under reversed cyclic loads. In some cases when the results of the three walls were not within 10% of one another a fourth test was performed. Two walls per configuration were tested with only a lateral load (one monotonic & one cyclic) such that their performance could be compared directly with the remaining walls that carried both lateral and gravity loads.

The wall specimens were constructed of ASTM A653 (2002) steel studs with a minimum grade and thickness of either of 230 MPa and 1.09 mm or 340 MPa and 1.37 mm. All studs had nominal dimensions of 92.1 mm (web) x 41.3 mm (flange) x 12.7 mm (lip). ASTM A653 top and bottom tracks with a minimum grade of 230 MPa and a thickness of 1.09 mm were used. The nominal dimensions were 92.1 mm (web) x 31.8 mm (flange). The chord studs consisted of two C-sections connected back-to-back with two No. 10-16 x 3/4" Hex head self-drilling screws at 305 mm o/c. This built-up member was used to prevent the flexural and/or local buckling failure of a single chord stud alone. The remaining interior studs were spaced at 610 mm o/c. Sheathing consisted of either 12.5 mm CSA O121 Douglas fir plywood (DFP) (CSA O121, 1978), 12.5 mm CSA O151 Canadian softwood plywood (CSP) (CSA O151, 1978) or 11 mm CSA O325 oriented strand board (OSB) (CSA O325, 1992) rated 1R24/2F16/W24. The sheathing was screw connected to one side of the wall and oriented vertically. Industry standard Simpson Strong-Tie S/HD10 holddown connectors with 7/8" (22.2) mm anchor rods were attached to the chords. ASTM A325 (2003) bolts, 3/4" (19.1 mm) in diameter, were used as shear anchors. No. 8 x 1/2" wafer head self-drilling framing screws were used to connect the track and studs. No. 8 x 1-1/2" Grabber SuperDrive bugle head self-piercing sheathing screws and No. 8 x 1-1/4" Grabber SuperDrive bugle head self-drilling sheathing screws were used to affix the sheathing to the light gauge steel frames. The self-drilling screws were used for the 1.37 mm thick chord studs. The sheathing screws were installed 12.7 mm from the edge of each wood panel. The screw spacing / fastener schedule was 75 mm or 152 mm along the panel edges and 305 mm in the interior.

Table 1. Matrix of wall configurations and values used for capacity based design approach.

Parameter	Shear Wall Test Specimens				
	47[a]-A,B,C[c] 48[b]-A,B,C[c]	**49[a]-A,B[c],C,D 50[b]-A,B,C[c]**	**51[a]-A,B[c],C 52[b]-A,B,C[c]**	**53[a]-A,B,C[c] 54[b]-A,B,C[c]**	**55[a]-A,B[c],C,D 56[b]-A[c],B,C**
Panel Type	12.5 mm DFP	11 mm OSB	11 mm OSB	12.5 mm CSP	12.5 mm CSP
Fastener Schedule[d] (mm)	75/305	152/305	75/305	152/305	75/305
Nominal Design S_y[e] (kN/m)	24.5	10.6	21.6	11.0	20.6
Capacity Design S_y (kN/m)	29.4	12.7	25.9	13.2	24.7
Total Gravity Load (kN)	18	18	18	18	18
Probable Chord Force (kN)	80.7	40.0	72.2	41.2	69.3
Chord Stud Thickness (mm)	1.37	1.09	1.37	1.09	1.37
Chord Capacity[f] (kN)	115.2	67.1	115.2	67.1	115.2
Perforated Chord Capacity[f] (kN)	99.9	58.7	99.9	58.7	99.9

[a] Monotonic loading protocol, [b] CUREE reversed cyclic loading protocol, [c] Gravity load not applied, [d] 75/305 refers to the spacing of the sheathing screws along the panel perimeter and in the field, respectively., [e] Branston et al. (2006a), [f] Calculated with CSA S136 (2001) where ϕ = 1.0, $K_x = K_y = 0.9$, $K_t = 0.65$, $L_x = 2.440$ and $L_y = L_t = 2$ x edge screw spacing.

Capacity Design of Chord Studs

To predict the probable compression forces that would be subjected to the chord studs due to lateral seismic loading the ultimate shear capacity of the wall was estimated based on the nominal (ϕ = 1.0) shear yield strength (S_y) from the previous testing of similar walls (Branston et al., 2006a) (Table 1). By multiplying the nominal shear yield strength of the wall by the recommended overstrength of 1.2, and then accounting for the wall height, it was possible to determine the maximum capacity based compression force that would be applied to the chord studs at the time of wall failure. The S_y values obtained from Branston et al. were based on shear walls in which the desired sheathing connection failure mode controlled the performance of the test specimens. Furthermore, an 18 kN gravity load was then distributed to the studs in the wall in an attempt to replicate the dead, live and snow load forces that may be present at the time of a design seismic event. The gravity load was considered to be constant over the length of the chord stud; however, the compression force from the lateral load increased from zero at the wall top to a maximum at the base of the wall. Therefore, the total compression load on the chord varied approximately linearly from top to bottom. This force was compared with the nominal compression capacity of the double chord studs as calculated using CSA S136 (2001) (Table 1). The full chord capacity was compared with the force at the base of the wall, while the perforated chord capacity was compared with the force at the height of the first web knockout, i.e. ≈ 837 mm above the base of the wall. Hikita (2006) also carried out compression tests of sheathed chord studs and showed that the axial capacity could best be estimated by using effective length factors of $K_X = K_Y = 0.9$ and $K_T = 0.65$, and braced lengths of $L_X = 2440$ mm and $L_Y = L_T = 2s$, where s is equal to the screw spacing along the perimeter of the wall. The capacity design forces should also be considered for the selection of all other elements in the seismic force resisting system, including the shear anchors, holddowns, etc. Note, a chord stud thickness of 1.37 mm was needed for the walls with sheathing screws spaced at 75 mm (Table 1). The thickness of these studs was greater than that used in the previous tests by Branston (2004) because of the capacity design approach that was applied.

Shear Wall Testing

The combined lateral and gravity loading of the shear walls was made possible with the use of specially constructed reaction frame (Fig. 1). A ± 125 mm dynamic actuator and 250 kN load cell were used to apply the lateral displacement to the test walls. The gravity forces were applied through jacks placed at either end of the shear wall specimen below the lower members of the test frame. Threaded rods were connected to the hydraulic jacks and continued up through to the loading beam at the top of the wall. A servo controlled hydraulic system was connected to the actuator and jacks such that they could be controlled simultaneously by means of an MTS TestStar computer system. The actuator was placed in stroke control, while the jacks were placed under load control; and thus were able to maintain the 9 kN force each during the lateral protocols that were applied. The threaded rods were permitted to rotate through the use of half round sections as the top of the wall was displaced in-plane (see inset Fig. 1). The inclination of the rods, a result of the lateral displacement of the loading beam, subjected the walls to additional horizontal forces that were accounted for in the final analysis of the test data. Additional details can be found in Hikita (2006).

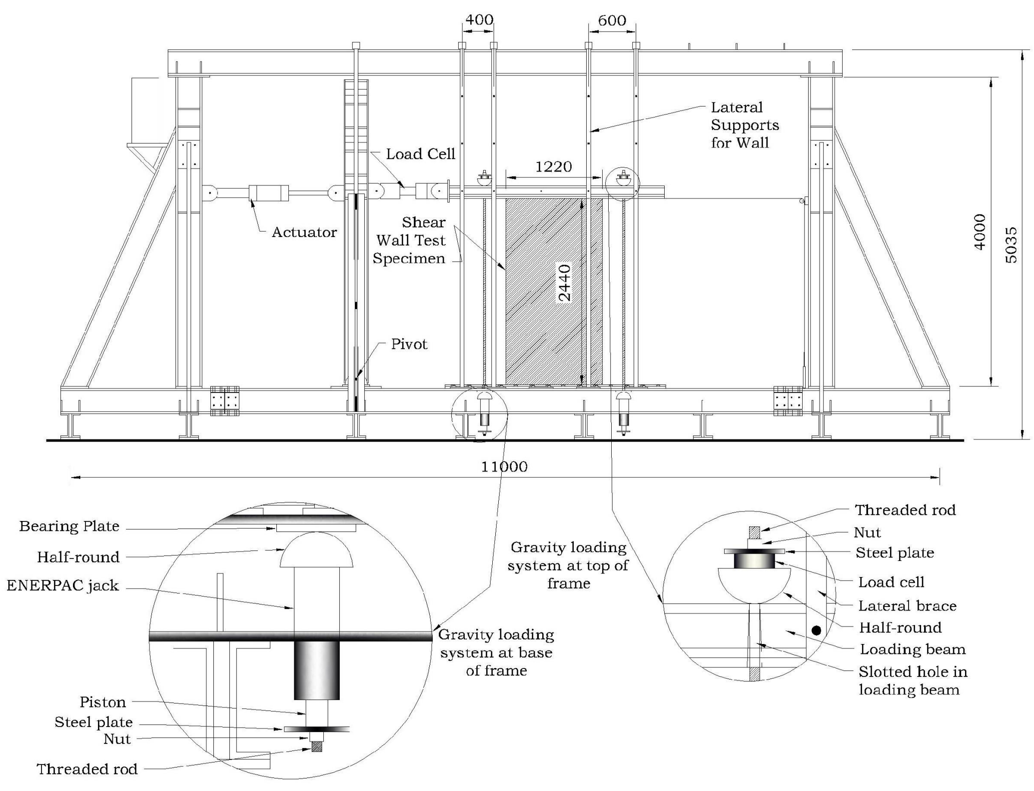

Figure 1. Combined gravity and lateral loading system used for shear wall testing.

Loading Protocols

A stroke controlled monotonic test protocol was used to displace the top of the wall at a constant rate of 7.5 mm/min. until failure. This protocol was similar to that used for previous shear wall tests at McGill University by Branston (2004), Chen (2004), Boudreault (2005), Blais (2006) and Rokas (2006), but did not include the evaluation of the permanent offset. The reversed cyclic tests were run using the CUREE protocol for ordinary ground motions (Krawinkler et al., 2000; ASTM E2126, 2005), which also was used for the previous shear walls tested for this study, as recommended by Boudreault. This protocol was developed using the results of non-linear time history dynamic analyses of structures constructed of wood frame shear walls. It is based on cumulative damage concepts, and accounts for the fact that multiple earthquakes may occur during the lifetime of a structure. The time history responses of the modeled buildings were converted to representative deformation controlled loading histories. The protocol subjects elements to ordinary ground motions (not near fault) with a probability of exceedance of 10% in 50 years. The displacement rate for the lateral actuator during cyclic loading was limited to a maximum of 10 mm/s due to the performance limitations of the hydraulic pump and oil supply for the two jacks. At small displacements in which the rate of loading was below this limit the displacement cycles were applied at a frequency of 0.5 Hz, as was done for all previous reversed cyclic tests at McGill University.

The level of gravity loads was selected based on a literature review of shear wall tests completed for both light wood frame and steel frame structures as well as an estimate for a typical multi-storey commercial structure constructed of cold-formed steel members. Gravity loads ranging up to 18.2 kN/m along the length of the wall have been utilized in the past to compensate for the lack of holddowns in wood walls (Ni & Karacabeyli, 2000). Durham et al. (2000) based the gravity load on an equivalent second storey above the test wall. Landolfo et al. (2006) also carried out tests of steel frame shear walls on which gravity loads were applied. The choice of a constant gravity load was made in view of the fact that steel frame / wood panel shear walls would typically incorporate holddowns, and based on the design of three-storey commercial structure with a typical snow load in the Vancouver region. Although the gravity load on a wall is highly variable depending on the occupancy, materials used and the floor spans it was felt that a line load of 14.8 kN/m, equivalent to 18 kN on a 1220 x 2440 mm wall, represented a conservative estimate of the potential vertical force expected during a seismic event.

Modes of Failure

The failure of all specimens was linked to the deterioration in load carrying capacity of the sheathing panel screwed connections, as was anticipated (Fig. 2). The bearing, plug shear, pull-through and screw shear failure modes were similar to those observed for the shear wall tests carried out by Branston (2004), Chen (2004), Boudreault (2005), Blais (2006) and Rokas (2006). Local buckling of the chord studs did not occur during any test, even when the additional gravity loads were applied. This sheathing connection failure was made possible by use of the capacity based approach in the design of the walls, and the resulting selection of 1.37 mm thick chord studs for the walls with a 75 mm sheathing screw spacing.

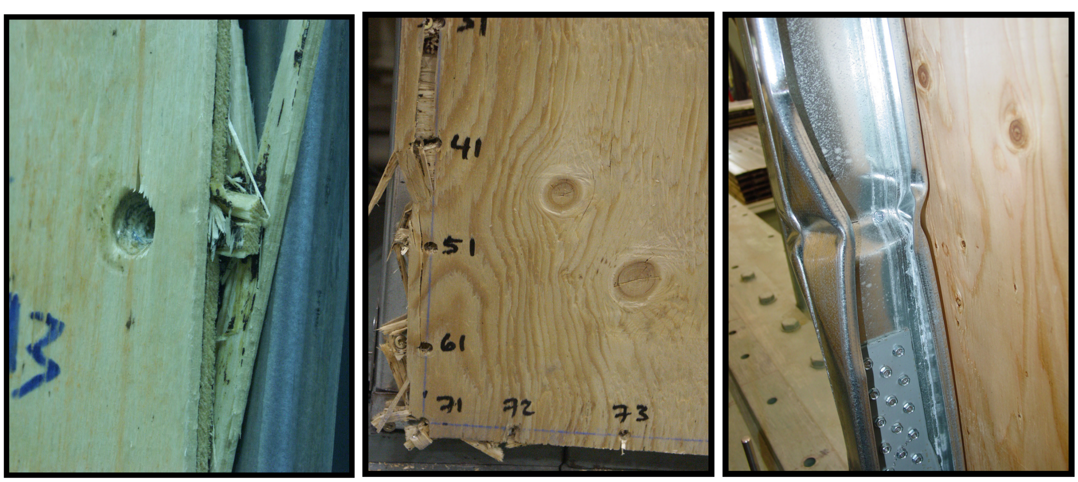

Figure 2. Sheathing connection failure (observed) and chord stud failure (not observed) (Branston, 2004).

Measured Performance

The interpretation of the test results followed the EEEP model as described by Branston et al. (2006a). A listing of the ultimate strength, S_u, nominal shear strength, S_y, post-peak displacement at $0.8S_u$, $\Delta_{0.8}$, ductility, $\mu = \Delta_{0.8}/\Delta_y$, where Δ_y is the displacement at first yield, K_e, and the energy for each test specimen is provided in Table 2. The cyclic test results represent the average of the positive and negative excursions. A comparison of the test results is presented in order to identify any possible impact on lateral load carrying capacity, stiffness and behaviour due to the application of combined gravity and lateral loads, even when sheathing connection failure occurs. It was typically not possible to identify which tests had undergone combined loading by viewing the test results without reference to the matrix of

Table 2. Design and measured values for shear wall specimens.

Test Specimen	S_y	S_u	K_e	μ	$\Delta_{0.8Su}$	E
	(kN/m)	(kN/m)	(kN/mm)	(mm/mm)	(mm)	(kN/mm)
Monotonic Tests						
47A	25.9	31.1	1.42	3.67	80.8	1573
47B	23.9	29.0	1.37	3.72	82.8	1465
47C[a]	26.1	32.4	1.40	3.41	74.4	1581
AVG.	25.3	30.8	1.40	3.60	79.3	1540
49A	9.88	10.9	1.69	8.36	60.1	668
49B[a]	10.5	11.7	1.40	5.77	52.7	617
49C	11.6	13.3	1.58	7.07	64.2	838
49D	10.4	12.1	1.64	6.73	52.2	614
AVG.	10.6	12.0	1.58	6.98	57.3	684
51A	20.3	22.2	2.25	4.96	54.5	1209
51B[a]	20.6	23.1	2.41	3.94	41.1	902
51C	20.4	22.4	2.78	5.43	48.5	1092
AVG.	20.4	22.5	2.48	4.78	48.0	1068
53A	11.4	13.4	1.02	5.66	77.4	973
53B	10.8	12.4	0.68	4.19	81.6	945
53C[a]	11.1	13.2	1.13	6.33	76.1	951
AVG.	11.1	13.0	0.94	5.39	78.4	956
55A	20.9	25.7	1.15	3.72	88.8	1272
55B[a]	22.8	28.4	1.23	3.16	75.9	1380
55C	20.7	24.7	1.06	3.35	83.9	1239
56D	21.3	27.1	1.10	3.22	84.1	1277
AVG.	21.5	26.5	1.14	3.36	83.2	1292
Reversed Cyclic Tests						
48A	25.5	28.6	1.58	4.28	58.7	3546
48B	25.0	28.2	1.75	4.46	57.5	3470
48C[a]	25.6	28.1	1.45	3.31	57.4	3700
AVG.	25.4	28.3	1.59	4.02	57.9	3572
50A	9.96	10.7	1.50	6.85	52.1	1247
50B	9.53	10.3	1.71	8.04	51.1	1185
50C[a]	10.1	10.9	1.61	7.25	49.1	1287
AVG.	9.87	10.6	1.60	7.38	50.7	1240
52A	20.7	21.5	1.68	2.57	35.3	1565
52B	20.1	22.2	2.53	3.72	33.0	1543
52C[a]	22.2	24.3	1.97	3.70	44.0	2382
AVG.	21.0	22.7	2.06	3.33	37.4	1830
54A	10.4	11.4	1.02	5.67	54.3	1654
54B	10.8	11.9	0.93	5.02	66.1	1698
54C[a]	10.9	12.3	1.15	4.86	52.8	1349
AVG.	10.7	11.9	1.03	5.18	57.7	1567
56A[a]	21.9	24.2	1.31	3.65	63.8	3488
56B	21.4	24.1	1.49	3.97	63.8	3149
56C	20.6	23.0	1.42	4.04	60.8	3115
AVG.	21.3	23.8	1.40	3.89	62.8	3251

[a] Gravity load not applied to shear wall.

wall specimens. The monotonic graph for test 49D (combined loading) and 49B (lateral loading) of wall resistance versus deformation in Fig. 3 are similar in general shape up to the peak load, after which they diverge only slightly. The hystereses of the reversed cyclic tests were also quite similar to one another within each configuration. Fig. 4 shows the hysteresis for test 50A (combined loading) and 50C (lateral loading), as well as the bilinear EEEP curves that are used for design. The backbone curves, created by joining the peaks of the loops for the maximum successive cycles, are also very similar in shape for the two tests. Test 50A reached slightly higher displacements, $\Delta_{0.8}$, while test 50C experienced slightly higher shear strength, S_y, (Table 2). These small variations in measured wall properties are not outside the range expected for nominally identical test specimens that are subjected to the same loading protocol. Previous testing has indicated some degree of variability exists in the measured performance from one test specimen to another even if they are considered to be nominally identical in terms of construction and loading. This variability may be attributed to the natural variation in the material properties of a particular wood species, a variation in placement (location and quality) of sheathing fasteners, pre-existing damage at sheathing connection locations, etc. Note, if the chord studs had not been designed properly using the capacity based approach, then it is possible that their failure would have occurred, especially for the walls with a 75 mm sheathing spacing. This would have affected the behaviour of the walls and the general shape of the response curves. Since the failure of all test specimens was consistently found to be in the sheathing connections, the wall behaviour was also consistent.

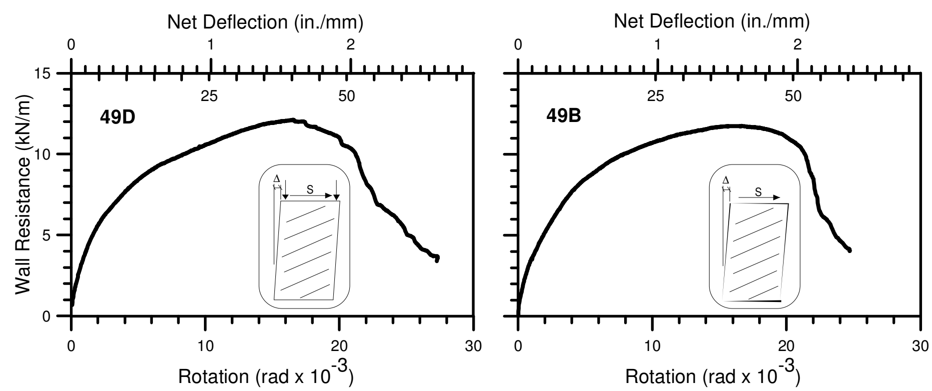

Figure 3. Monotonic lateral load vs. deformation test curves with and without gravity loads.

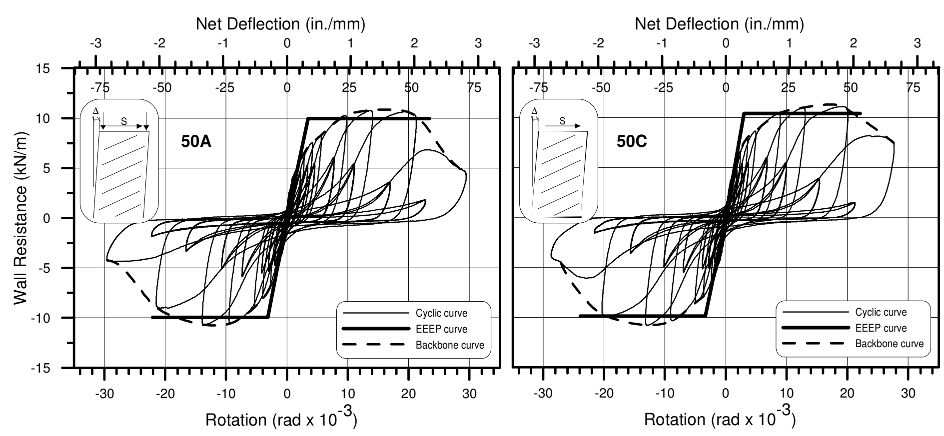

Figure 4. Reversed cyclic lateral load vs. deformation test curves with and without gravity loads.

Table 3 contains a list of the measured test results (S_u, $\Delta_{0.8}$, E) and EEEP design parameters (S_y, μ, K_e, E) normalized to the values obtained for the nominally identical wall specimen (configuration and loading protocol) for which gravity loads were not applied. The ultimate shear resistance, S_u, reached during testing was slightly higher for the specimen without gravity loads compared to the majority of those subjected to both gravity and lateral forces, however the range of normalized values was 0.87 to 1.13. The displacement at $0.8S_u$ and the dissipated energy were almost always higher for the shear walls that had to resist a gravity and lateral load. Only for test 52A and B did $\Delta_{0.8}$ fall measurably below that of the wall without gravity loads. The direct measurements taken during testing indicate that a marginal decrease in ultimate shear capacity was observed as well as a marginal increase in the ability to displace and dissipated energy when gravity loads were applied.

Table 3. Normalized measured properties and design values of shear wall specimens.

Test Specimen	47 Series[a]	48 Series[b]	49 Series[a]	50 Series[b]	51 Series[a]	52 Series[b]	53 Series[a]	54 Series[b]	55 Series[a]	56 Series[b]
Ultimate Resistance, S_u										
A	0.96	1.02	0.93	0.98	0.96	0.88	1.02	0.93	0.91	1.00[c]
B	0.89	1.00	1.00[c]	0.94	1.00[c]	0.91	0.94	0.97	1.00[c]	1.00
C	1.00[c]	1.00[c]	1.13	1.00[c]	0.97	1.00[c]	1.00[c]	1.00[c]	0.87	0.95
D	N/A	N/A	1.03	N/A	N/A	N/A	N/A	N/A	0.95	N/A
Displacement at $0.8S_u$										
A	1.09	1.02	1.14	1.06	1.32	0.80	1.02	1.03	1.17	1.00[c]
B	1.11	1.00	1.00[c]	1.04	1.00[c]	0.75	1.07	1.25	1.00[c]	1.00
C	1.00[c]	1.00[c]	1.22	1.00[c]	1.18	1.00[c]	1.00[c]	1.00[c]	1.11	0.95
D	N/A	N/A	0.99	N/A	N/A	N/A	N/A	N/A	1.11	N/A
Dissipated Energy (Test)										
A	1.10	1.20	1.12	1.06	1.35	0.92	1.05	1.04	1.15	1.00[c]
B	1.05	1.10	1.00[c]	1.01	1.00[c]	1.07	1.03	1.09	1.00[c]	0.95
C	1.00[c]	1.00[c]	1.40	1.00[c]	1.22	1.00[c]	1.00[c]	1.00[c]	1.03	0.89
D	N/A	N/A	1.01	N/A	N/A	N/A	N/A	N/A	1.09	N/A
Yield Resistance, S_y										
A	0.99	0.99	0.94	0.98	0.98	0.93	1.02	0.95	0.92	1.00[c]
B	0.91	0.98	1.00[c]	0.94	1.00[c]	0.90	0.97	0.99	1.00[c]	0.98
C	1.00[c]	1.00[c]	1.11	1.00[c]	0.99	1.00[c]	1.00[c]	1.00[c]	0.91	0.94
D	N/A	N/A	0.99	N/A	N/A	N/A	N/A	N/A	0.94	N/A
Ductility, μ										
A	1.08	1.29	1.45	0.94	1.26	0.70	0.89	1.17	1.18	1.00[c]
B	1.09	1.35	1.00[c]	1.11	1.00[c]	1.01	0.66	1.03	1.00[c]	1.09
C	1.00[c]	1.00[c]	1.23	1.00[c]	1.38	1.00[c]	1.00[c]	1.00[c]	1.06	1.11
D	N/A	N/A	1.17	N/A	N/A	N/A	N/A	N/A	0.96	N/A
Stiffness, K_e										
A	1.01	1.09	1.21	0.93	0.93	0.85	0.90	0.88	0.93	1.00[c]
B	0.98	1.21	1.00[c]	1.06	1.00[c]	1.28	0.60	0.80	1.00[c]	1.13
C	1.00[c]	1.00[c]	1.13	1.00[c]	1.15	1.00[c]	1.00[c]	1.00[c]	0.86	1.08
D	N/A	N/A	1.17	N/A	N/A	N/A	N/A	N/A	0.89	N/A
Dissipated Energy (EEEP)										
A	0.99	0.96	1.08	0.97	1.34	0.66	1.02	1.23	0.92	1.00[c]
B	0.93	0.94	1.00[c]	0.92	1.00[c]	0.65	0.99	1.26	1.00[c]	0.90
C	1.00[c]	1.00[c]	1.36	1.00[c]	1.21	1.00[c]	1.00[c]	1.00[c]	0.90	0.89
D	N/A	N/A	1.00	N/A	N/A	N/A	N/A	N/A	0.93	N/A

[a] Monotonic test series [b] Reversed cyclic test series [c] Gravity loads not applied to shear wall.

A comparison of the test parameters that would typically be used in design, i.e. are part of the EEEP analysis procedure, show similar findings. The shear yield resistance, S_y, for the combined loading tests was somewhat lower than for the walls with only a lateral load. The range of the normalized ratio was from 0.90 to 1.11. The ductility measurement, μ, typically favoured the tests with gravity loads except in a few isolated cases. The elastic stiffness, K_e, of the shear walls without gravity loads was reasonably close to that of the walls with both gravity and lateral loads; however, no consistent trend with respect to which loading type provided for higher stiffness was observed. The dissipated energy values, obtained from the area under the EEEP curve were also reasonably close for the test with and without gravity loads.

The average S_y values obtained from all of the tests by Hikita (2006) were similar to those recommended by Branston et al. (2006a) for use in design, e.g. Hikita obtained S_y = 21.4 kN/m (CSP), 25.3 kN/m (DFP) and 20.7 kN/m (OSB) for the walls with 75 mm spaced screws, whereas Branston et al., recommended S_y = 21.6 kN/m (CSP), 24.5 kN/m (DFP) and 20.6 kN/m (OSB). The stiffness values, K_e, obtained by Hikita were similar to the design values listed by Branston et al. Calibration of the test-to-predicted shear strengths following the method outlined in the AISI Specification (2004) for limit states designed resulted in a resistance factor of $\phi = 0.7$, which is consistent with that recommended by Branston et al. Furthermore, the seismic force modification factors determined using the procedure described by Mitchell et al. (2003) were $R_o = 1.71$ and $R_d = 2.89$, which confirm those recommended by Boudreault et al. (2007) ($R_o = 1.70$ and $R_d = 2.50$) which were based on the results of physical tests and non-linear time history dynamic analyses.

Conclusions

A comparison of the measured shear wall deformation, energy and strength parameters indicates that there was no consistent or distinct influence on the lateral load carrying performance of steel frame / wood panel shear walls due to the inclusion of gravity loads. In some cases the gravity loads improved the measured parameters, whereas in others the gravity loads caused the opposite to occur; for example, the ultimate shear resistance was slightly lower for the walls with a gravity load compared with that recorded for the walls subjected to lateral loads alone. Much of the variation in the measured parameters can likely be attributed to a variation in the material properties and construction from one wall specimen to another rather than the application of a gravity loads. The final shear strength and stiffness design values were very similar to those recommended by Branston et al. (2006a), which were based on a separate set of test data. The design parameters derived from the 32 shear wall specimens also support the recommendation of seismic modification factors of $R_d = 2.5$ and $R_o = 1.7$ by Boudreault et al. (2007) and $\phi = 0.7$ by Branston et al.. Failure in all walls was at the sheathing connection locations; chord stud failure did not occur. These outcomes, however, are dependent on the fact that the chord studs were designed following a capacity based approach, whereby failure of the sheathing connections was forced to occur. If an inappropriately sized chord stud were selected, because the ultimate shear load of the wall were incorrectly predicted or if gravity companion loads were not considered, then it would be possible for local buckling of the stud members to take place. In this instance a degradation of the strength, deformation capacity (ductility) and energy dissipation capability of the walls may be observed when gravity loads are imposed with lateral loads. Nonetheless, if steel frame / wood panel shear walls are properly designed and constructed, such that chord stud failure is avoided, then the lateral performance of the walls can be represented by shear wall tests in which lateral loads alone are applied.

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