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Monotonic and cyclic behaviour of wood frame shear walls for mid-height timber buildings

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ABSTRACT

The Chilean forestry industry has a significant presence in the economy of the country. Due to pollution problems and the high seismicity of the region, timber is a suitable material for new buildings. However, because of cultural customs and high demands of regulations, nowadays it is difficult to construct buildings higher than three stories in Chile. However, several international projects have shown that is feasible to construct mid-height timber buildings. Sturdy end studs and strong hold-downs are needed in mid-height wood buildings (up to 6 stories high) to resist large vertical and horizontal loads. However, design parameters provided by current seismic design provisions for those shear walls do not consider the effects of sturdy end studs and strong hold-downs in lateral strength and stiffness of the walls. In order to address this issue, a multidisciplinary team at the Catholic University of Chile has conducted an extensive experimental and numerical research program. This paper presents the results of seventeen in-plane monotonic and cyclic shear tests in wood frame shear walls of different lengths (1200, 2400 and 3600 mm) and 2470 mm height. The walls have five 2 × 6″ end studs, strong hold-down anchorages and standard 11.1 mm structural OSB panels (1200 × 2400 mm) on both faces of the wall and with nails in the edge of the OSB panels spaced at 50 or 100 mm. The main objectives of this research are to evaluate the seismic response of these shear walls and to assess the current code expressions applied to shear walls with sturdy end studs to be used in mid-height timber buildings. The results show that, while cyclic loads reduce the monotonic shear strength of walls, cyclic loads do not influence the ultimate displacement and stiffness. The main benefits of a smaller nail spacing are the increase of the strength and delay of stiffness degradation. The unit shear was influenced by wall length: 1200 mm walls presented a better unit shear capacity than 2400 and 3600 walls, and there were not observable differences between 2400 and 3600 mm walls. The characteristic damping of the walls varied between 7 and 10%. Finally, the current design provisions underestimate the shear strength and overestimate the stiffness of walls to be used in mid-height timber buildings.

1. Introduction

Wood frame structural elements have been widely studied to develop or to improve the structural design methods of wood construction [1–7]. Research about wood frame buildings has strongly focused in the response of shear walls and horizontal diaphragms. The research on shear walls has been mainly oriented to low-height residential structures; the configuration of the typical wall studied is shown in Fig. 1. A typical wood frame shear wall consists of a 1200 or 2400 mm long frame structure, with 2 × 4″ studs typically spaced at 400 mm on centres, double end studs, and 2 × 4″ single plates at the top and bottom of the wall. The walls are usually sheathed with 11-mm OSB or plywood panels on the exterior face and may have a gypsum panel on the interior face of the wall. The spacing of the nails along the edges of the panels may be 50, 75, 100, or 150 mm. The walls have hold-down anchors to prevent overturning. To establish an accurate yet practical design method for timber frame shear walls under in-plane shear forces, Griffits [1] obtained test data from a large number of shear wall tests with sheathing panels of various materials and formulated two empirical design methods. Richard et al. [2] and Williamson and Yeh [3] analysed and tested shear walls with window openings, typical of wood frame dwelling structures. They proposed methods for a more accurate prediction of cyclic response of the shear walls in order to develop a solution to properly design wood frame shear walls, providing the walls

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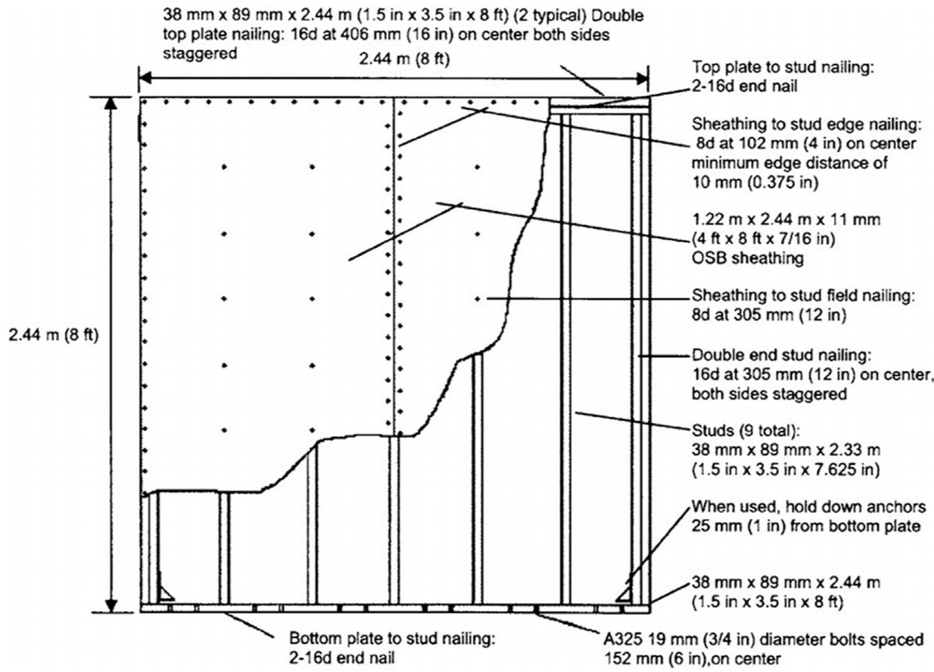

Fig. 1. Typical shear wall specimen (Lebeda et al. [14]).

with adequate strength and stiffness.

Results from several researches have been used to update the design expressions and values of stiffness and strength of the Special Design Provisions for Wind and Seismic (SDPWS) [8], which provides mechanical properties of materials and requirements for design and construction of wood members, fasteners and assemblies to resist wind and seismic forces. Eighty walls were tested by Line et al. [4] to define the nominal unit shear capacities, among other properties, used for design.

To obtain a representative data set to support a design methodology, Salenkovich and Dolan [5] tested typical wood shear walls with various aspect ratios under monotonic and cyclic in-plane shear loads, using as sheathing material standard OSB panels. NAHB Research Center [6] performed similar testing, but with fiberboard as sheathing material. Rosowsky et al. [7] tested similar walls to examine the effect of washer size, and no significant differences in performance were observed. The focus of the researches has been mainly to improve the quality of the characterization of the shear wall performance under different loading and obtain information of the wall components, namely: sheathing material, joints, studs, hold-downs, shear bolts, among others.

Dishongh and Fowler [9] studied the effects of door and window openings in walls with gypsum sheathing on two wall faces and concluded that the lateral response of a wall with an opening in the centre could be analysed as two separated shear walls, disregarding the length of the openings. Kamiya et al. [10] evaluated the effect of wall length on lateral resistance of shear walls with aspect ratios that varied between 1/3 and 2, with plywood sheathing, concluding that the shear strength was directly proportional to wall length. Toothman [11] compared the lateral response of cyclic and monotonically loaded walls, and also investigated the contribution of gypsum panels to walls with different sheathing materials on the opposite face of the walls. He also, studied the effects of including hold-downs compared to not using them. He concluded that, in general, cyclic loads produce a decrease in the performance indicators, like strength and ultimate displacement. Gypsum panels produced a significant increase on the overall strength, elastic stiffness and energy dissipation of the walls, but those effects cannot be added linearly to the response of walls without gypsum panels. The walls without hold-downs, had in average 66% smaller peak load than walls with hold-downs. The effect of large sheathing panels is a theme of interest for the typical configuration of shear walls. Lam et al. [12] studied the lateral resistance of shear walls with nonstandard large OSB sheathing panels (2.4×7.3 m) and standard size OSB sheathing panels (1.2×2.4 m). It was observed that walls with nonstandard large sheathing panels had a significant increase of stiffness and strength, 36% and 30%, respectively. The ductility ratios did not vary significantly for both dimensions of panels. The walls with standard size panels dissipated more energy than walls with large panels under cyclic loads. Durham et al. [13] tested twelve 2.4×2.4 m walls with large (2.4×2.4 m) and standard (1.2×2.4) size OSB panels. Shear strength and initial stiffness increased 26% and 30% respectively when large panels were used. On the other hand, walls with large panels had approximately 25% smaller maximum drift than walls with standard OSB panels.

As mentioned before, hold-down anchorages are a vital component for wood frame shear walls. Lebeda et al. [14] studied the consequences of placing the hold-down anchors at different positions in thirteen typical wood frame shear walls, concluding that placing hold-downs at the first interior stud instead of the end stud have decremental effects on the structural performance of wood shear walls: average strength decreased 42% in monotonic tests and 35% in cyclic tests. Non-typical failure modes were detected when the hold-downs were placed at the first interior stud, namely, hold-down fastener failure, splits in the bottom plate, vertical studs separated from the top and bottom plates and significant reduction in wall energy dissipation occurred. Twenty-one shear walls with hold-down anchors and shear walls without hold-down anchors were tested by Johnston et al. [15], some of them with simultaneous uniform vertical load applied. They concluded that the lateral stiffness and the energy dissipation capacity increased when a vertical load is applied. Hold-down anchors have little effect on lateral strength, stiffness and energy dissipation of the walls with vertical loads between 12 kN/m and 25 kN/m. They also concluded that shear wall design procedures at that time (2006) were conservative for walls subjected to vertical loads and that more efficient designs could be realized if vertical loads were considered in the design procedures. The 2015 SDPWS [8] does not consider the favourable effects of vertical loads in the lateral response of shear walls.

Nowadays mid-height timber buildings in the USA are designed using the current edition of the SDPWS [8], where properties and requirements for wind and seismic design are detailed for wood members, fasteners and assemblies. A performance-based design method for wood frame buildings was presented by Pang and Rosowsky [16], where a direct displacement design procedure is used to carry out the design. The behaviour of the shear walls has been characterized with non-linear modelling using CASHEW program [17] for commonly used shear wall configurations, in which was assumed that framing members are rigid elements with pin-ended connections that do not contribute to the lateral response of the walls.

Mid-height timber buildings require a shear wall configuration that allows them to resist large vertical and horizontal loads. The typical shear wall configuration has not enough vertical strength for this kind of buildings. The research done to develop the current design methodologies has considered the standard wood frame wall configuration used in residential structures, like the one shown in Fig. 1. In this paper, we refer to strong hold-downs as hold-downs that have larger dimensions and are stronger than the ones used by previous researchers. Investigations on the lateral response of walls with sturdy end studs and strong hold-downs (typically needed in this type of buildings) are scarce: Pei and Van de Lindt [18] presented a coupled shear-bending model for analysis of stacked wood shear walls. They verified the model with the data of shear tests of a three-story shear wall with continuous rod hold-downs, and two 2 × 6″ studs on each side of the rods. Van de Lindt et al. [19] tested in a shake table a full scale six-story light-frame wood building with walls with sturdy end studs and continuous ATS rods. The building performed very well in all tests, suffering only nonestructural damage. Marzaleh et al. [20] studied wood frame shear walls with strong hold-downs and sturdy end studs, but subjected to vertical loads and bending moment in addition to monotonic lateral load, so it is difficult assess the effects of only the strong hold-downs and sturdy end studs on the lateral response. Three tests were executed and analytical models were generated and was concluded that the existing analytical methods underestimate the shear stiffness of the tested walls. Construction details at the anchorages were highly influential on the shear strength measured.

The main objectives of this paper are to investigate the seismic response of wood frame shear walls with sturdy end studs and strong hold-downs to be used in mid-height buildings, and to evaluate how well the current SDPWS [8] expressions fit with the measured behaviour of the tested walls.

As part of a larger project to evaluate the changes that have to be introduced in the current Chilean seismic design code NCh433 [21] to allow for multi-storey construction using wood, 17 wall specimens with large vertical strength were tested: seven walls were subjected to monotonic in-plane shear load and ten were subjected to cyclic in-plane shear load. An analysis of the experimental results is presented in this paper to determinate the properties and behaviour of the walls with sturdy end studs and strong hold-downs in terms of ultimate load, ductility, stiffness and damping. Strengths and wall deflections were evaluated and compared with code expressions and results obtained by other authors.

2. Materials and methodology

2.1. Test specimens

Specimens with the configuration shown in Fig. 2 were tested under in-plane shear loading. The walls were 2470 mm height with three different lengths: 1200, 2400 and 3600 mm. The wood frame of the walls consisted of MGP10 graded Chilean radiata pine elements (NCh1198 [22]), which is similar to Southern Pine: 2 × 6″ vertical studs spaced at 407 mm, as well as 2 × 6″ double plates at the top and bottom of the wall. The 2 × 6″ elements are 36 × 138 mm. The sturdy end studs consist of five vertical studs (see Figs. 2 and 3). Double vertical studs were placed at the edges of 1200 by 2400 11.1-mm thick APA rated OSB sheathing panels [23] mounted on both faces of the wall for ease of nailing the panels to the wood frame. Nails ϕ3 × 70 mm, spaced at 50 or 100 mm, were used to join the OSB panels to the frame elements. Simpson-StrongTie HD12 hold-down anchors were bolted with four ϕ1 × 10″ bolts to the sturdy end studs, and with one ϕ1–1/8 × 10″ bolt to the foundation, as shown in Fig. 3. To prevent sliding of the wall, ϕ1 × 10″ shear bolts were installed through the bottom plate, between vertical studs.

A list of the test specimens, their dimensions and nail spacing are shown in Table 1. The alphanumeric code used to identify the specimens indicates the loading protocol (M for monotonic, C for cyclic), length of the wall in centimetres, spacing of the edge nails in centimetres, and specimen number.

2.2. Test set up

The hold-downs of each specimen were bolted to a steel foundation beam. The lateral load was applied by a hydraulic actuator and distributed uniformly to the wall through a steel plate bolted to the top of the wall. The walls were horizontally braced by two steel rods attached to the steel plate on the top of the wall and fixed to a reaction concrete wall to prevent out-of-plane displacements. Transducers were used to measure the lateral displacement of the top of the wall at the level of the hydraulic actuator, slip of the wall respect to the foundation beam, deformation of the diagonals of the wall, and uplift in exterior edge of the walls. The test setup of the specimens is shown in Fig. 4.

2.3. Test procedure

The CUREE loading protocol [24] was used to test the walls. Displacement controlled monotonic and cyclic tests were executed. The parameters obtained from the monotonic tests were used to calibrate the cyclic loading protocol. Fig. 5 shows the normalized displacement of the CUREE cyclic loading test protocol. This protocol was applied in all cyclic tests, which were executed up to failure of the walls.

3. Results and discussion

The main results presented in this section are the observed failure modes of the tested walls; hysteresis curves which were constructed using the total lateral displacement; strengths and displacements reached by the walls; calculated properties like characteristic equivalent damping, elastic stiffness and ductility. Envelope curves obtained from the cyclic tests and monotonic curves are compared to observe the effect of wall length, nail spacing and cyclic loads on the performance of the walls. Finally, the experimental results are compared with the estimations of strength and stiffness from SDPWS [8] provisions, to see how well the code expressions fit with the measured behaviour of the walls.

3.1. Failure mode

The walls presented failure modes similar to walls tested by authors like Shenton et al. [25], Lebeda et al. [14], Johnston et al. [15], among others. The failure mode observed in all the walls was due to the cutting of sheathing nails (Fig. 6a), pull out of the nails in the edges of the wall sheathing (Fig. 6b) and crushing of OSB panel by nails head (Fig. 6c). The failure of the walls commonly occurred by a combination of the three mentioned modes. The frame structure remained undamaged or slightly damaged; also, the connection zone between the hold-downs and the end studs did not show any local damage. Nevertheless, in two tests the frame structure of the walls experienced considerable damage at very large lateral displacements. In the case of wall C120-05-02 the bottom plate broke in its entire length (Fig. 7a); in wall M120-05-02 the zone of the connection between the bottom plate and the end studs was extensively damaged (Fig. 7b) and the end studs failed in tension (Fig. 7c) at a very large lateral displacement.

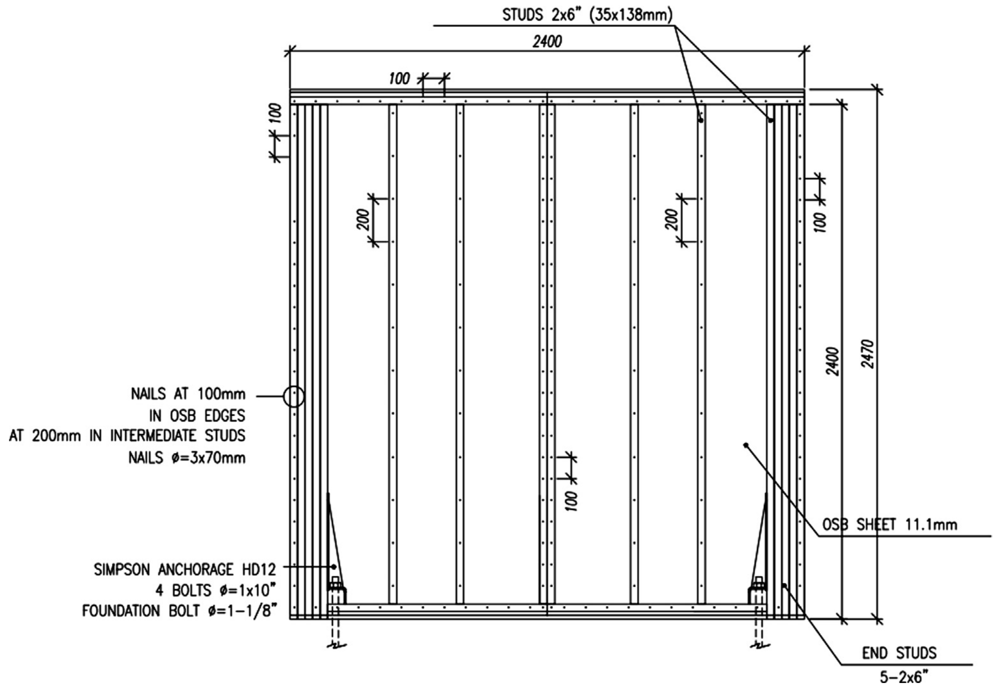

Fig. 2. Configuration of a 2400 mm shear wall (lengths in millimetres).

Fig. 3. Sturdy end studs hold-down anchorage.

Table 1
List of tested specimens.

Shear wall	Loading	Length [mm]	Nail spacing [mm]
M120-10-01	Monotonic	1200	100
M120-10-02	Monotonic	1200	100
M120-05-01	Monotonic	1200	50
M120-05-02	Monotonic	1200	50
M240-10-01	Monotonic	2400	100
M240-10-02	Monotonic	2400	100
M240-05-01	Monotonic	2400	50
C120-10-01	Cyclic	1200	100
C120-10-02	Cyclic	1200	100
C120-05-01	Cyclic	1200	50
C120-05-02	Cyclic	1200	50
C240-10-01	Cyclic	2400	100
C240-10-02	Cyclic	2400	100
C240-05-01	Cyclic	2400	50
C240-05-02	Cyclic	2400	50
C360-10-01	Cyclic	3600	100
C360-10-02	Cyclic	3600	100

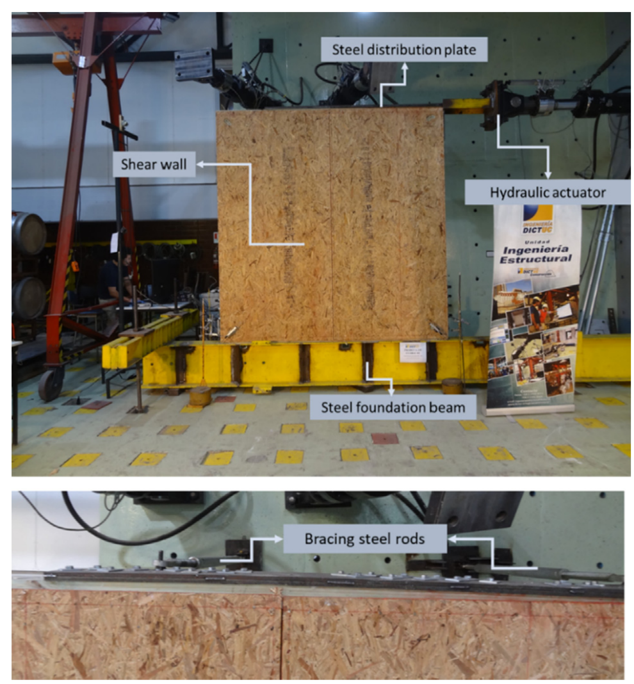

Fig. 4. Test setup.

3.2. Force-displacement response and main results

The hysteresis curves of five configurations of walls are shown in Fig. 8. The shape of the curves of the walls is similar to those reported by other authors for typical wall configurations [4,15,25]. From these hysteretic response curves the envelope curves for positive and negative displacement of the top with respect to the bottom of the wall were obtained. Initially, the response is approximately linear up to drift values of 0.5–0.8%. After that, the response is highly nonlinear due to the local deformation of the studs and OSB panels at the connections with the nails. The former results in strong hysteretic pinching effect. As the length of the walls increase the post peak strength decreases more

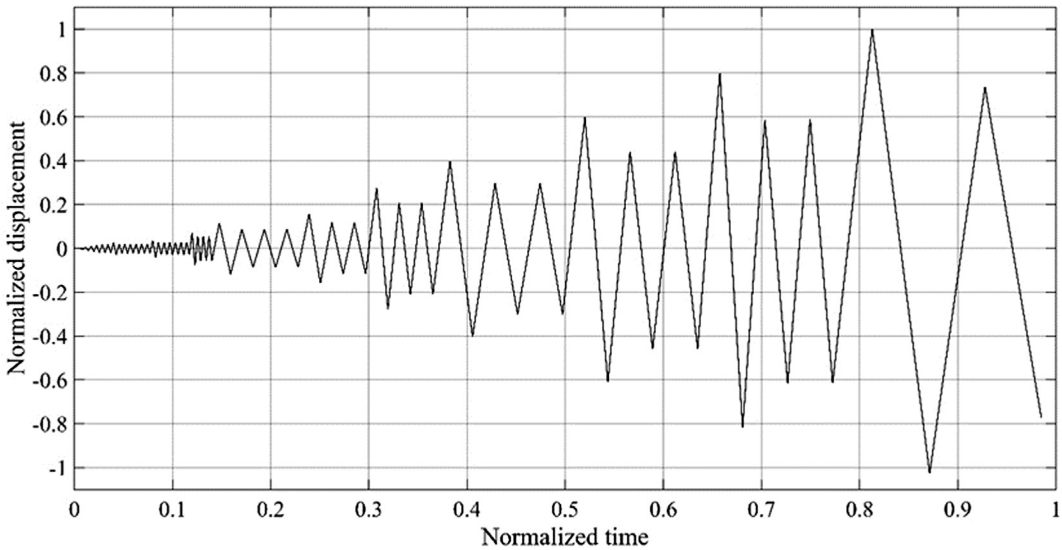

Fig. 5. CUREE loading history for cyclic load test.

rapidly.

In Table 2, are summarized the maximum load measured on the tests (P_{max}), secant stiffness at 40% P_{peak} (K_0), drift level at 40% P_{peak} (δ_{40}), characteristic equivalent viscous damping ratio (ξ), yield and ultimate displacements (Δ_y and Δ_u) and ductility ratio (μ). P_{peak} is the maximum load of the average of the two envelope curves of each cyclic test.

From Table 2 and Fig. 8 it is possible to see that the influence of wall length on the wall performance is considerable. Also, the nails spaced at 50 mm produce an increase in the maximum load of the walls as compared to nails at 100 mm; on the other hand, ultimate displacement is similar in all the cyclic tests, independent of wall length and nail spacing.

3.3. Measured strength

The maximum loads of the walls with 50 mm nail spacing were larger than for walls with 100 mm nail spacing. The average maximum measured loads of the 1200 mm long walls with nails at 50 mm under cyclic load were 27% higher than walls with nails at 100; for walls under monotonic loads the maximum loads were in average a 39% larger than walls under cyclic loads. For the 2400 mm long walls with nails at 50 mm under cyclic load were 17% higher than walls with nails at 100; for walls under monotonic loads the maximum loads were in average a 11% larger than walls under cyclic loads. It is observed that the effect of nail spacing is smaller as the length of the walls increase.

The walls tested under cyclic loads have smaller maximum load than the walls tested under monotonic loading. The decrease of strength varied between 7 and 16%, which is consistent with results obtained by Toothman [11], who concluded that cyclic loading reduced the peak load in average by 12%.

In Table 3 are shown the average unit strengths of the walls, to assess the effect of the wall length on the strength of the walls. For walls under cyclic loading, unit strength is approximately 10% larger for 1200 mm long walls than for longer walls. For the case of monotonic loading, the difference increases to at least 37%. It is important to notice that cyclic loading produced a decrease of unit strength of at least 27% for 1200 mm long walls, while for 2400 mm long walls that decrease of strength was of only 10%.

Wall M240-10-02 was not considered in the previous analysis because this wall presented unusual small strength, even smaller than the strength of the corresponding walls subjected to cyclic loading. The cause of this could not be identified.

3.4. Deformation

The yield and ultimate displacements were calculated from the tests, in addition to the drift at 40% of peak value of the envelope curve.

(a) Cutting of sheathing nails (b) Pull out of the nails (c) Crushing of OSB sheet by nails head

Fig. 6. Failure mode of the walls.

(a) (b) (c)

Fig. 7. Damage in bottom plate and end studs.

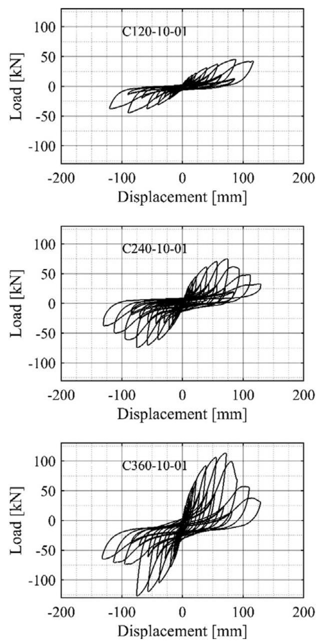

Fig. 8. Typical hysteretic response for cyclic tests. Displacement of the top with respect to the bottom of the wall.

Table 2
Summary of results from the tests.

Tested wall	P_{max} [kN]	K_0 [kN/mm]	δ_{40}	ξ	Δ_y [mm]	Δ_u [mm]	μ
C120-10-01	45.3	1.11	0.0065	0.07	35	116	3.3
C120-10-02	43.5	0.93	0.0072	0.10	38	116	3.0
C120-05-01	56.3	1.25	0.0071	–	39	115	3.0
C120-05-02	56.8	1.22	0.0075	0.10	40	148	3.7
C240-10-01	75.0	2.62	0.0046	–	24	87	3.6
C240-10-02	77.5	2.85	0.0041	0.10	22	89	4.1
C240-05-01	86.0	2.80	0.0050	0.10	27	87	3.2
C240-05-02	92.5	3.60	0.0040	0.08	22	91	4.2
C360-10-01	127.1	4.85	0.0040	0.09	21	82	3.9
C360-10-02	114.6	3.90	0.0046	0.10	24	78	3.2
M120-10-01	50.4	1.12	0.0073	–	39	189	4.8
M120-10-02	47.1	1.22	0.0063	–	33	151	4.6
M120-05-01	66.0	1.01	0.0105	–	54	160	3.0
M120-05-02	69.2	0.96	0.0117	–	64	163	2.5
M240-10-01	86.8	2.76	0.0051	–	28	134	4.8
M240-10-02	70.5	2.25	0.0051	–	27	87	3.2
M240-05-01	96.4	2.85	0.0055	–	30	111	3.7

Ultimate displacement Δ_u is the measured displacement of the wall at failure, unless the corresponding ultimate load P_u is less than 0.8 P_{peak}, in which case Δ_u is calculated as the displacement associated to 0.8 P_{peak} (see Fig. 9). The yield displacement was calculated from the equivalent energy elastic-plastic (EEEP) curve according to ASTM

Table 3
Unit strength for different wall lengths (kN/m).

Loading	Nail spacing [mm]	Wall length [mm]		
		1200	2400	3600
Cyclic	100	37.0	31.8	33.6
	50	47.1	37.2	–
Monotonic	100	40.6	36.1	–
	50	56.3	40.2	–

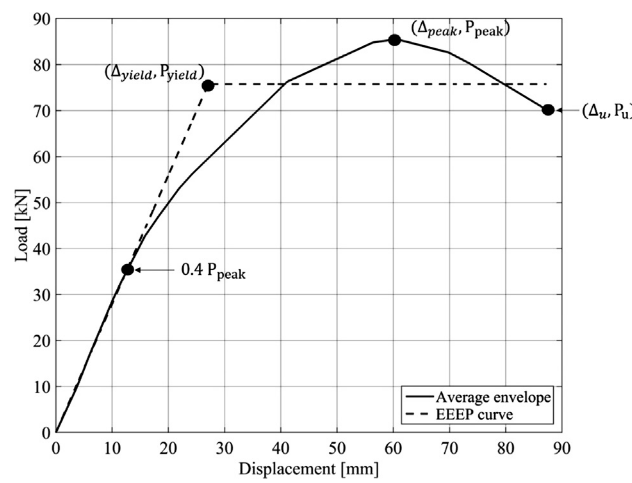

Fig. 9. Average envelope and EEEP curve of wall C240-05-01.

E2126-11 [26]. The EEEP curve is an elastic-plastic curve with the same area enclosed by an envelope curve, with the elastic part defined at 40% P_{peak}. Average envelope curves were calculated for each hysteresis curves of the cyclic test, while for the monotonic tests the measured load-displacement curve was used directly. An EEEP curve and an envelope curve are shown in Fig. 9. The values of Δ_y and Δ_u are listed in Table 2.

The yield displacement Δ_y varied between 22 and 40 mm (Table 2), which corresponds to a drift approximately between 0.9 and 1.7%. Ultimate displacement Δ_u varied between 80 and 170 mm, which corresponds to a drift of approximately between 3 and 7%. Walls subjected to monotonic loading reached larger values of ultimate displacement than walls under cyclic loads: drift levels between 4.5 and 7.0%, while for cyclic tests the ultimate drift varied between 3.2 and 4.7%. Similar trend is observed for the yield displacements.

The yield displacement Δ_y and ultimate displacement Δ_u decreased as the wall length increased from 1200 mm to 2400 mm, but those values were similar for wall lengths of 2400 and 3600 mm. No trend was observed in the effect of the nail spacing on the ultimate displacements.

The drift level at 40% P_{peak} (δ_{40}) is calculated as the lateral displacement reached at that load level, normalized by the height of the wall. δ_{40} varies between 0.4 and 0.8% for cyclic walls and between 0.5% and 1.2% for monotonic walls. At these levels of drifts walls should be in elastic regime, undamaged.

The ductility ratio is calculated as $\mu = \Delta_u/\Delta_y$ and values varied between 2.5 and 4.8. Ductility ratio was in average 3.75, with a coefficient of variation of 0.19. No trend was observed in the influence of nail spacing or wall length on ductility ratio. The decrease of the yield displacement associated to the increase of wall length is smaller than the decrease of the ultimate displacement, resulting that the 2400 long walls had a ductility ratio greater than the 1200 walls.

Walls M120-05-01 and M120-05-02 were excluded of the analysis of yield displacement because irregularly large values of yield displacement were measured for these walls.

3.5. Equivalent viscous damping

The equivalent viscous damping ratio ξ_{eq} (EVD) is a parameter calculated for walls under cyclic loads, to assess the capacity of the wall to dissipate energy and allows to estimate the damping ratio when the wall is considered as an external damper. To calculate EVD of the walls, it is necessary to subtract the rocking displacement from the measured lateral displacement of the walls, because this displacement generates an energy dissipation that depends of the interaction between the hold-down and the wood frame elements. After rocking displacement is subtracted, the EVD calculated does not depends of the anchorage system. EVD is obtained from the ratio of the hysteretic energy dissipated in the tests to the energy dissipated by a viscous damper in a sinusoidal loop with the same displacement amplitude as the hysteretic cycle. The energy dissipated by a viscous damper is calculated as the area enclosed by the triangles limited by points of maximum displacement of each cycle. Fig. 10 illustrates a loop of a hysteretic curve and the parameters for calculating the damping ratio. EVD for cycle i is calculated as follows:

$$\xi_{eq,i} = \frac{E_{H,i}}{2\pi(0.5\ \ d_i^+P_i^+ + 0.5\ \ d_i^-P_i^-)} \tag{1}$$

where $E_{H,i}$ is the hysteretic energy dissipated by the wall in the cycle i, calculated as the area enclosed by the curve of the cycle, and - di+, di− are the maximum displacements of the cycle i, corresponding to loads Pi^+, P_i^-.

The characteristic value of EVD (ξ) was calculated as the 10% percentile of EVD of all the cycles. In Fig. 11 are shown typical plots of EVDs of the cyclic tests and the characteristic value of EVD as a horizontal line. The characteristic value of EVD varied between 7 and 10%, with an average value of 9% and a standard deviation of 1%. The obtained values are in typical ranges for wood frame shear walls reports

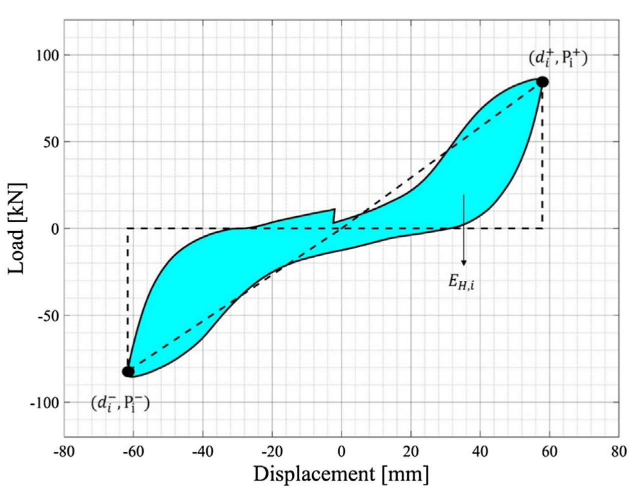

Fig. 10. Illustration of a loop for damping calculation.

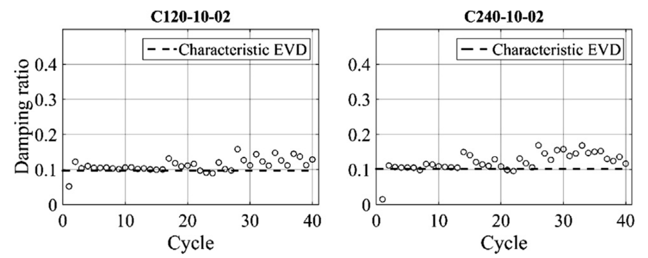

Fig. 11. Equivalent viscous damping ratio of cyclic tests and characteristic EVD.

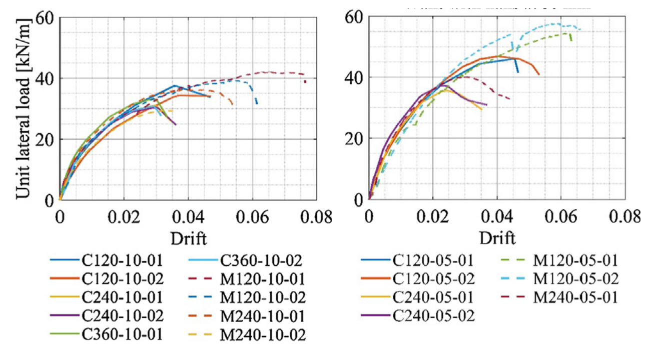

Fig. 12. Unit lateral loads vs drift: envelope curves.

reviewed by Jayamon et al. [27].

3.6. Envelope response and stiffness

Fig. 12 shows unit shear loads ν_{exp} (measured load divided by the length of the wall) versus drift curves of the monotonic tests and envelope curves of the hysteretic response of the walls under cyclic loads. In Tables 4 and 5 are presented the measured unit shear strength ν_{exp}, the unit shear strength calculated using SDPWS [8] ν_{SDPWS}, the ratio of calculated to measured unit shear strengths, the secant unit stiffness at 40% P_{peak} (K_{40}), the unit stiffness K_{SDPWS} calculated using the SDPWS [8] expression for deflection and the ratio of K_{40} to K_{SDPWS}.

The behaviour of almost all the unit shear envelope curves and monotonic curves was very similar up to a drift of 0.4–0.8%. At larger values of drift, the general shape of the unit envelope curves depends mostly on the nail spacing. On the other hand, as mentioned before, the maximum lateral displacement depends of the length of the wall.

The unit stiffnesses was calculated at 40% P_{Peak} of the load-displacement curves, which corresponds to a drift level of 0.4–0.8% in

Table 4
Unit load capacity and stiffness of walls with nails at 100 mm.

Specimen	ν_{exp} (kN/m)	ν_{SDPWS} (kN/m)	ν_{exp}/ν_{SDPWS}	K_{40} (kN/mm/m)	K_{SDPWS} (kN/mm/m)	K_{40}/K_{SDPWS}
M120-10-01	42.0	20.4	2.06	0.93	1.19	0.79
M120-10-02	39.2	20.4	1.92	1.01	1.19	0.85
C120-10-01	37.8	20.4	1.85	0.93	1.19	0.78
C120-10-02	36.3	20.4	1.78	0.78	1.19	0.66
M240-10-01	36.1	20.4	1.77	1.15	1.84	0.62
M240-10-02	–	20.4	–	0.94	1.84	0.51
C240-10-01	35.2	20.4	1.53	1.09	1.84	0.59
C240-10-02	32.3	20.4	1.58	1.19	1.84	0.65
C360-10-01	35.3	20.4	1.73	1.35	2.16	0.62
C360-10-02	31.8	20.4	1.56	1.08	2.16	0.50
Average	35.5		1.75	1.04		0.63
Standard dev.	3.7		0.18	0.16		0.10
CV	0.10		0.10	0.16		0.16

Table 5
Unit load capacity and stiffness of walls with nails at 50 mm.

Specimen	ν_{exp} (kN/m)	ν_{SDPWS} (kN/m)	ν_{exp}/ν_{SDPWS}	K_{40} (kN/mm/m)	K_{SDPWS} (kN/mm/m)	K_{40}/K_{SDPWS}
M120-05-01	55.0	34.1	1.61	0.84	1.45	0.58
M120-05-02	57.6	34.1	1.69	0.80	1.45	0.55
C120-05-01	46.9	34.1	1.37	1.04	1.45	0.72
C120-05-02	47.3	34.1	1.38	1.02	1.45	0.70
M240-05-01	40.2	34.1	1.18	1.19	2.56	0.46
C240-05-01	35.8	34.1	1.05	1.16	2.56	0.45
C240-05-02	38.5	34.1	1.13	1.50	2.56	0.58
Average	45.9		1.34	1.08		0.58
Standard dev.	8.3		0.24	0.24		0.10
COV	0.19		0.18	0.22		0.18

walls under cyclic loading and drift levels of 0.5–1.2% in walls under monotonic loading. While the measured unit stiffness of the walls with nails spaced at 100 mm varied between 0.78 and 1.35 kN/mm/m, with an average value of 1.04 kN/mm/m, the unit stiffness of the walls with nails at 50 mm are slightly larger and varied between 0.80 and 1.50 KN/mm/m, with an average value of 1.08. There is no statistical difference between the stiffness of both types of walls. Therefore, at drift levels less than 0.8% (approximately equivalent to load levels less than 40% of peak) the lateral stiffness does not depend on the nail spacing, but mainly on the length of the walls.

At larger values of drift, the stiffness degradation of walls with nails at 100 mm occurs faster than in walls with nails at 50 mm. After a 1% drift it is possible to clearly differentiate the responses of walls with different nail spacing. The above is verified in detail in Fig. 13.

In Fig. 14 are shown the load displacement curves of walls with different lengths and nails spaced at 100 mm, where is observed the effect of wall length on the performance of the walls. As the wall length changes from 1200 to 2400 mm, the maximum unit load and displacements present a considerable decrease. However, when the wall length changes from 2400 to 3600 the walls present very similar maximum unit load, ultimate displacements and the behaviour of the walls did not vary significantly. The same trend can be observed for the walls with nails spaced at 50 mm.

4. Design considerations

A comparison between measured values in the tests and SDPWS [8] provisions is presented in the current section. Strength, stiffness and

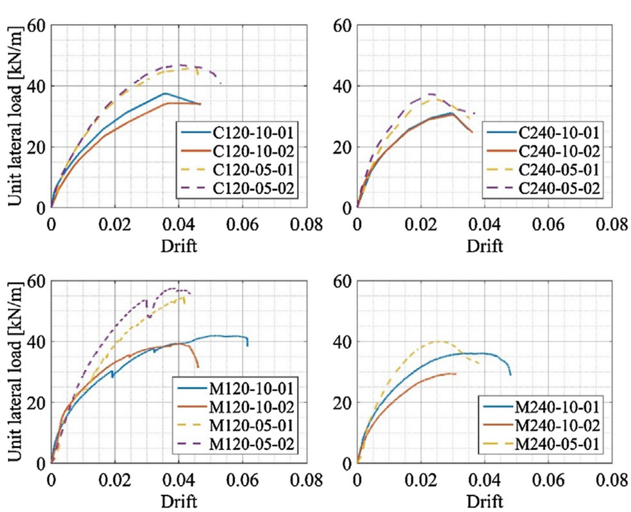

Fig. 13. Effect of nail spacing on performance of the walls.

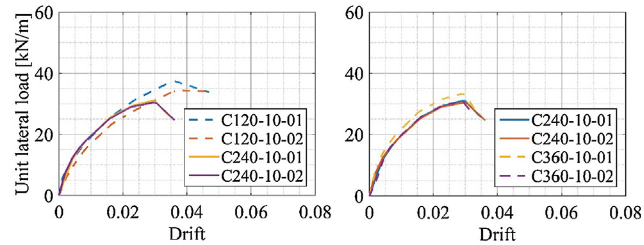

Fig. 14. Effect of wall length on performance of the walls.

deflections were evaluated to make the comparison. The main objective of this section is to quantify the differences between the provisions and the measured behaviour to evaluate if provisions are appropriate to design wood frame shear walls with sturdy end studs and strong hold-downs, or if it is necessary to propose new provisions for this kind of walls.

4.1. Design values

The mechanical properties of the radiata pine wood used in the tested walls have similar mechanical properties to the wood that is specified SDPWS [8], namely Southern Pine. Also, OSB panels and nails used in the walls correspond to the ones to be used with SDPWS [8]. Therefore, to calculate the strengths and design stiffness of the tested walls, SDPWS [8] values were used directly. In Tables 4 and 5 are presented the measured and calculated unit shear strengths and stiffness.

4.1.1. Strength

The values of the calculated unit shear strength (ν_{SPDWS}) were obtained from Table 4.3A of the SDPWS [8]. 11 mm (7/16 in) OSB panels were considered on both faces of the wall and common 8d nails were used. For nails spaced at 100 mm (4 in) the calculated unit shear capacity of the walls is 20.4 kN/m (1400 plf), while for nails spaced at 50 mm (2 in) the calculated unit shear capacity is 34.1 kN/m (2340 plf). In all cases the measured strength is larger than the design strength. The average ratio of measured to calculated strength is 1.75 and 1.34 for walls with nails spaced at 100 mm and 50 mm, respectively. For walls under cyclic loading only, those ratios are 1.67 and 1.23, respectively.

In Table 6 is shown a comparison between average experimental unit shear strengths and code calculated strengths of typical shear walls tested under cyclic loading by other authors [4,14,15,25]. The shear walls were 2.4×2.4 m (Fig. 1), with one 11 mm OSB panel and 100 mm nail spacing. The ratios of measured to calculated strengths are less than 1.44, and the average ratio is 1.30, smaller than the corresponding value of the walls tested in this research, which is 1.67.

4.1.2. Stiffness and deflections

The stiffness of the walls can be calculated from SDPWS [8] Eq. (4.3-1), used to calculate by elastic analysis the lateral displacement. The horizontal inter-story displacement δ_{ws} used to design, is calculated as (Eq. (2)):

Table 6
Average values of unit strength of typical tested shear walls.

Test	Strength (kN/m)	SPDWS Strength (kN/m)	Strength ratio
Line et al. [4]	13.5	10.2	1.32
Lebeda et al. [14]	13.3	10.2	1.30
Johnston et al. [15]	14.7	10.2	1.44
Shenton et al. [25]	11.7	10.2	1.15
Average	13.2	10.2	1.30

$$\delta_{ws} = \frac{8\ \nu h^3}{EAb} + \frac{\nu h}{n\ G_a} + \frac{h\ \ \Delta_a}{b} \tag{2}$$

where:

ν = unit lateral load (kN/mm)
h = wall height = 2470 mm
E = modulus of elasticity of the wood from Table 4.b Nch1198 [22] = 10.000 MPa
A = area of end studs = 138 * 36 * $N_{end\ studs}$ = 4968 mm^2 * $N_{end\ studs}$
b = wall length (mm)
n = number of sheathed wall faces = 2
G_a = modulus of shear of OSB panels from Table 4.3.A SDPWS = $\begin{cases} 7.355(nails\ \ at\ \ 50\ \ mm) \\ 3.853(nails\ \ at\ \ 100\ \ mm) \end{cases} \frac{\text{kN}}{\text{mm}}$
D_a = vertical elongation of wall anchorage system (mm)
$N_{end\ studs}$ = number of end studs

This expression for deflection includes bending, shear, and rocking components of displacement. The displacement Δ_a was obtained from force balance of the wall. If the hold-down stiffness K_{hd} and the force in the hold-down T_{hd} are known, it is possible to calculate Δ_a as T_{hd}/K_{hd}. The force T_{hd} can be calculated from the force balance shown in Fig. 15, while K_{hd} is obtained from Simpson StrongTie C-C-2017 catalogue [28] and reached a value of 13.7 kN/mm for the used hold-down. Eqs. (3)–(6) show the calculations of the unit stiffness of the walls K_{SDPWS}.

$$\Delta_a = \frac{Ph}{K_{hd}b'} \tag{3}$$

$$\delta_{ws} = \frac{8\ \nu h^3}{EAb} + \frac{\nu h}{1000\ \ n\ \ G_a} + \frac{h}{b}\frac{Ph}{K_{hd}b'} \tag{4}$$

$$\nu = \frac{P}{b} \tag{5}$$

Then, it is possible to calculate the wall stiffness as:

$$K_{SDPWS} = \left(\frac{8\ \ h^3}{EAb^2} + \frac{h}{b\ \ n\ \ G_a} + \frac{h^2}{K_{hd}bb'}\right)^{-1} \tag{6}$$

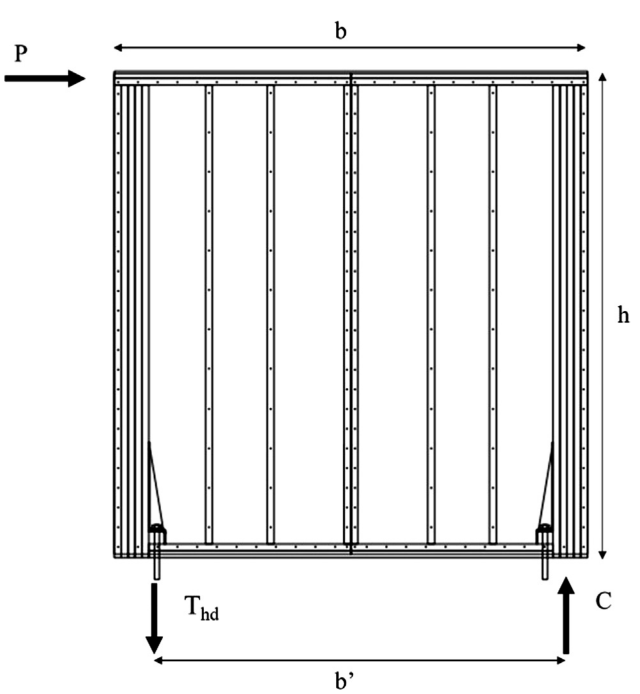

Fig. 15. Diagram of the overturning forces on the wall.

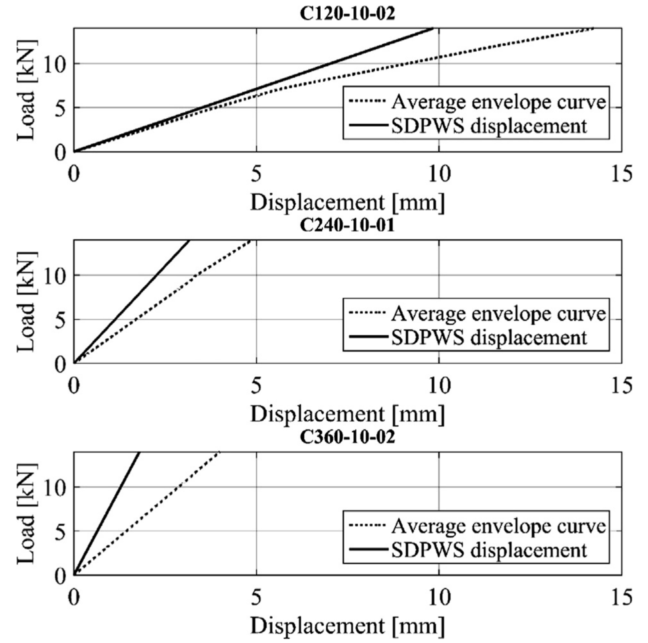

Fig. 16. Measured and calculated load-displacement curves for loads levels than 40%P_{peak} of walls with nails at 100 mm.

Load-displacement curves were calculated for wall C240-05-01, C240-10-01 and C360-10-02 using the previous expression and are plotted in Fig. 16, together with the corresponding experimental curves for displacements not larger than 15 mm (0.61% drift). It is possible to see that the measured initial stiffness is larger than the calculated stiffness for walls 2400 mm and 3600 mm long, while for 1200 long walls the stiffness was similar.

The unit stiffness measured at 40% P_{Peak} are smaller than the stiffness calculated from Eq. (4.3-1) of SDPWS [8], with an average ratio of measured to calculated stiffness of 0.68 for walls with nails at 100 mm and 0.67 for walls with nails at 50 mm (see Tables 4 and 5).

Shear deformation (δ_{shear}) was measured in all the tests by means of diagonal transducers that measure the diagonal deformation of each wall (δ_1). It was possible to calculate the shear stiffness (K_{shear}) of each wall using the procedure illustrated in Fig. 9. Average envelope curves were calculated for each P-δ_{shear} hysteresis curve. K_{shear} is calculated at 40% of P_{peak} with corresponding value of δ_{shear} of the envelope curve. Then, it is possible to calculate an experimental apparent shear stiffness (G_a) of the shear wall using the following expression provided by the SDPWS [8] (see Fig. 17):

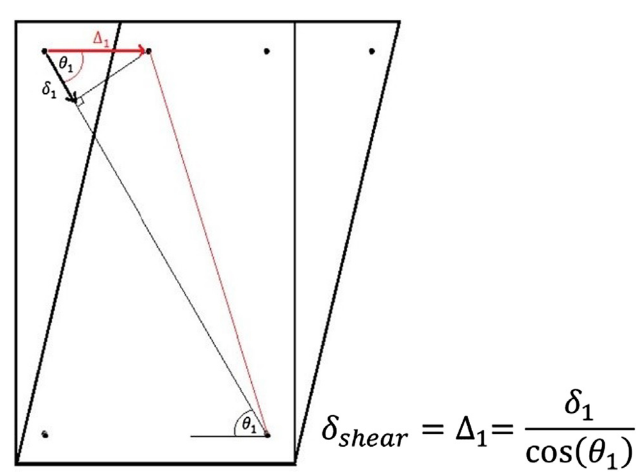

Fig. 17. Shear component of wall deflection.

Table 7
Apparent shear stiffness of walls with nails at 100 mm.

Wall (nails at 100 mm)	G_a [kN/mm] SDPWS	G_a [kN/mm] Experimental	Ratio
M120-10-01	3.9	2.4	0.63
M120-10-02	3.9	–	–
C120-10-01	3.9	2.4	0.63
C120-10-02	3.9	2.3	0.60
M240-10-01	3.9	2.3	0.59
M240-10-02	3.9	1.9	0.51
C240-10-01	3.9	2.3	0.60
C240-10-02	3.9	2.0	0.53
C360-10-01	3.9	2.2	0.58
C360-10-02	3.9	2.0	0.52
Average		2.2	0.58
Standard dev		0.17	0.04
CV		0.08	0.08

$$G_a = \frac{P}{\delta_{shear}}\frac{h}{nL} \rightarrow \boldsymbol{G_a = K_{shear}\frac{h}{nL}} \tag{7}$$

This value of G_a is compared with the value of G_a provided by Table 4.3A of the SDPWS [8] and the difference are observed in the Tables 7 and 8. It is observed that measured G_a values are in average 40–50% smaller than the values obtained from Table 4.3A of the SDPWS [8]. These differences produce an underestimation the lateral deformations of the tested walls.

Wall M120-10-02 was not considered in the previous analysis because this wall presented irregularities in the measurement of displacements.

4.2. Design implications

The code expressions overestimate the lateral stiffness of the walls and underestimate the strength of the walls. Stiffness overestimation may be because the G_a values provided by the SDPWS [8] are too large, as it is observed in Tables 7 and 8. In Tables 4 and 5 is observed that theoretical strengths were in average 1.74 and 1.38 times larger than observed strengths for walls with nails at 100 mm and 50 mm respectively. The dissimilarity between the two ratios indicates that values of strength are inadequate for an accurate design. Furthermore, the results presented in Tables 4 and 5 do not consider vertical loads effects. The tested walls are oriented to mid-height building, thus, most of the walls will be subjected to vertical loads that should increase the strength even more.

The expression for wall deflection of the code provides an inaccurate prediction of the real behaviour of the measured lateral deflection. If the SDPWS [8] expression is used in the walls, stiffnesses are overestimated and smaller deformations are obtained. The above could produce that actual drifts in structures can be larger than expected.

Codes for seismic design usually have drift limits to prevent damage or excessive deformations and to limit P-delta effects. In Chile, NCh433 [21] limits the drift calculated using an elastic analysis to 0.2% of the height of the wall, mostly to improve earthquake resilience of structures. The previous value was calibrated from the response of reinforced concrete walls buildings in large earthquakes in Chile. Timber structures are more flexible, so that drift limit may be too restrictive. Table 9 shows the measured loads (as percentage of maximum load) at different drift values (0.2%, 0.3%, 0.4% and 0.5%), taken from the measured envelope curves of cyclic tests of these wood frame walls and concrete walls tested by Alarcón et al. [29] (M/Vd ratio 2.5), DICTUC [30] (M/Vd ratio 1.0) and Amón [31] (M/Vd ratio 2.5). Concrete walls had different longitudinal and transverse reinforcing steel ratios. The obtained results were independent of the reinforcing steel ratios.

Table 8
Apparent shear stiffness of walls with nails at 50 mm.

Wall (nails at 50 mm)	G_a [kN/mm] SDPWS	G_a [kN/mm] Experimental	ratio
M120-05-01	7.4	3.0	0.40
M120-05-02	7.4	3.4	0.47
C120-05-01	7.4	2.9	0.40
C120-05-02	7.4	3.2	0.44
M240-05-01	7.4	5.5	0.75
C240-05-01	7.4	2.8	0.39
C240-05-02	7.4	3.9	0.54
Average		3.5	0.48
Standard dev		0.94	0.13
CV		0.27	0.27

Table 9
Load levels (as percentage of maximum shear load) measured at different drift values of wood frame and concrete walls.

	Load at drift 0.2%	Load at drift 0.3%	Load at drift 0.4%	Load at drift 0.5%
Concrete walls				
Average level	42%	54%	64%	74%
standard dev.	9,7%	8,8%	7,8%	8,7%
Walls with nails at 100 mm				
Average level	23%	32%	38%	44%
standard dev.	3.6%	3.4%	2.8%	3.1%
Walls with nails at 50 mm				
Average level	16%	25%	33%	39%
standard dev.	5.9%	6.8%	7.7%	7.9%

At drift level of 0.5% of wood frame shear walls, load levels were similar to the load levels reached by reinforced concrete walls at drift levels of 0.2%. Thus, it is concluded that wood frame shear walls allow higher drift levels than reinforced concrete walls, and a limit drift level of 0.2% used in current limit of the Chilean Seismic Design Code may be too conservative in terms of strength for the wood frame walls studied in this paper. A reasonable value of drift limit for mid-height timber buildings could be 0.4%, levels at which loads were less than 40% the maximum load.

5. Conclusions

Seventeen wood frame shear walls with sturdy end studs and strong hold-downs were tested under monotonic and cyclic loads. The seismic response of walls with sturdy end studs and strong hold-downs was investigated and current design provisions were assessed. The studied walls are to be used in mid-height buildings. The main conclusions of this investigation are as follows:

- At failure of the walls, the damage concentrated mainly in the sheathing-frame joints as nails were cut, sheathing crushed and nails pulled out. The frame structure remained mainly undamaged.
- The characteristic equivalent viscous damping of the walls was very similar among the walls, and varied between 7% and 10%, with an average characteristic value of 9%.
- The average ductility ratio was 3.75 and varied between 2.5 and 4.8. It was observed that nor nail spacing nor wall length influence the ductility ratio.
- The 1200 mm long walls reached larger values of ultimate and yield displacements than longer walls.
- Ultimate displacement is influenced by wall length. It was observed that 1200 mm walls have greater ultimate displacement than 2400 mm walls; however, there were no differences between 2400 mm and 3600 mm walls.
- Cyclic loading produced a decrease of ultimate and yield displacements.
- The strength depends of wall characteristics and there are aspects of the wall or load history that influence them. Cyclic loads produced a

decrease between 8 and 16% of shear strength with respect to the monotonic loading. Walls with nails at 50 mm had larger strength than walls with nails at 100 mm, as it was expected.
- Stiffness depends mainly of wall length. There were no differences in the initial stiffness of walls subjected to cyclic or monotonic loading. Also, the stiffness measured at 40% P_{peak} was similar for both values of nail spacing. The benefit in stiffness due to nail spacing was a slower decrease of stiffness at large drift levels (larger than 0.8%).
- Shear strength calculated using SDPWS [8], underestimates the strength of the walls, while the stiffness is overestimated. It may be necessary to use new expressions or testing results for the design of large-scale wood frame structures, or at least for wood frame shear walls with sturdy end studs and strong hold-downs.
- Apparent shear stiffness (G_a) provided by SDPWS [8] is the principal cause of the overestimation of lateral displacements because the measured value of G_a was lower than the theoretical value of the apparent shear stiffness.
- A drift limit larger than the current limit of the Chilean Seismic Design Code could be used to design wood buildings. A drift limit of 0.4% is proposed, which corresponds to undamaged response of wood frame walls.

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