PONTIFICIA UNIVERSIDAD CATOLICA DE CHILE

ESCUELA DE INGENIERIA

SEISMIC RESPONSE OF WOOD FRAME SHEAR WALLS WITH STURDY END STUDS AND STRONG HOLD-DOWN ANCHORAGES AND DESIGN IMPLICATIONS

FELIPE DAVID GUÍÑEZ YÁÑEZ

Thesis submitted to the Office of Research and Graduate Studies in partial fulfillment of the requirements for the Degree of Master of Science in Engineering

Advisor:

HERNÁN SANTA MARÍA OYANEDEL

Santiago de Chile, (October 2018)

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A mis Padres, Nancy e Iván, a mis hermanos y a mis amigos que siempre confiaron en mí y me dieron su apoyo para continuar este camino.

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RESUMEN

Diecisiete muros construidos en sistema marco-plataforma de diferentes largos (1200, 2400 y 3600 mm) y de 2470 mm de alto, fueron ensayados para conocer su comportamiento sísmico por medio de la aplicación de cargas cíclicas y monotónicas de corte en el plano. Los muros tienen cinco pies derechos de borde de 2x6", con anclajes hold-down de alta resistencia y paneles estructurales OSB de 11.1 mm de espesor (1200 x 2400 mm) en ambas caras del muro con clavos en el borde de los paneles espaciados a 50 o 100 mm. Los principales objetivos de esta investigación son evaluar la respuesta sísmica de estos muros y observar qué tan bien se ajustan las expresiones de un código de diseño usado hoy en día con el comportamiento medido de los muros con pies derechos de borde robustos, que podrían ser usados en muros de maderas de mediana altura. Los resultados indican que las cargas cíclicas producen una disminución en la resistencia al corte y desplazamiento último de los muros, pero la rigidez no fue influenciada por las cargas cíclicas. La rigidez calculada al 40% de P_{peak}, fue levemente mayor en muros con clavos cada 50 mm en los bordes de los paneles de OSB y el principal beneficio de un menor espaciamiento de clavos fue una degradación de rigidez más lenta y un aumento en la resistencia. Los muros de 1200 mm presentaron una mejor capacidad de corte unitario que los muros de 2400 y 3600 mm y no hubo diferencias observables entre los muros de 2400 y 3600 mm. Un amortiguamiento característico fue calculado, que fue en promedio un 9% y varió entre 7 y 10%. Las propiedades de los muros de corte entregadas por las ediciones actuales del Intenational Building Code y el código Seismic Design Provisions for Wind and Seismic no consideran los efectos de pies derechos robustos y hold-downs de alta resistencia en la resistencia y rigidez de los muros, por lo tanto, este tipo de muros no son apropiadamente caracterizados, y los procesos de diseño generan diseños demasiado conservadores o imprecisos para muros usados en edificios de madera de mediana altura.

Palabras clave: muros marco-plataforma, muros de corte, resistencia al corte, rigidez, edificios de madera de mediana altura, pies derechos robustos, hold-downs de alta resistencia.

ABSTRACT

Seventeen wood frame shear walls of different lengths (1200, 2400 and 3600 mm) and 2470 mm height were tested under cyclic and monotonic in-plane shear loads. The walls have five 2x6" end studs, strong hold-down anchorages and standard 11.1 mm structural OSB panels (1200 x 2400 mm) on both faces of the wall and with nails in the edge of the OSB sheets 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 how well current code expressions (Special Design Provisions for Wind and Seismic, SDPWS) fit with the measured behaviour of shear walls with sturdy end studs, to be used in mid-height timber buildings (up to 6 stories high). The results showed that while, cyclic loads considerably reduce the monotonic shear strength of walls, the ultimate displacement and stiffness were not influenced by cyclic loads. The stiffness calculated at 40% P_{peak} was slightly larger in the walls with nails in the edges of OSB panels spaced at 50 mm than 100 mm; the main benefit of a smaller nail spacing was a slower stiffness degradation and the increase of the strength. 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. A characteristic damping of the wall was calculated and was obtained that the walls had in average a 9% and varied between 7 and 10%. Properties of shear walls provided by current editions of the International Building code and the Seismic Design provisions for Wind and Seismic do not consider the effects of sturdy end studs and strong hold-downs in lateral strength and stiffness of the walls, therefore, this kind of shear walls are not properly characterized, and the design procedures generate too conservative designs of walls to be used in mid-height timber buildings.

Keywords: wood frame, shear walls, shear strength, stiffness, mid-height timber buildings, sturdy end studs, strong hold-downs.

1 INTRODUCTION

1.1 Background and problematic

Typical wood frame shear wall configuration has been widely studied to develop or to improve design methods for wood structures that are widely used for residential structures. It is necessary to know the behaviour of structural components of wood frame buildings, and then, generate more accurate designs. However, mid-height timber buildings, require shear wall configurations able to resist high vertical loads. Capacity of typical shear wall configuration is not enough for this kind of buildings and investigations on the lateral response of walls with sturdy end studs and stronger hold downs is scarce. For this reason, a new configuration was studied, that fits the needed requirements of mid-height buildings.

Chile has large potential in wood resources, however, structural usage is less than classic materials like concrete and steel. The above is because research about wood structural components has not developed enough. The existent literature about wood frame elements is oriented mainly to residential structures. Thus, it is difficult to design and build multi-story buildings.

Nowadays, in Chile, there are not enough provisions to design mid-height timber buildings. This research was part of a large project to evaluate the changes that have to be introduced in the current Chilean seismic design codes to include provisions that allow the design of mid-height timber buildings.

1.2 Objectives

The main objectives of this research are to understand the seismic response of wood frame shear walls with sturdy end studs and strong hold-downs, and obtain an accurate characterization of the walls studied in this investigation through the calculation of their properties to explain the effect of different aspects of the walls, namely: wall length, nail spacing, cyclic loads and failure modes; and to evaluate how well code expressions fit with the measured behaviour of the tested walls. In this way, it is wanted to promote the wood as a construction material for multi-story buildings.

1.3 Literature review

Wood frame structural elements have been widely studied to develop or to improve the structural design methods of wood construction. Research about wood frame buildings has mainly focused in the response of shear walls and horizontal diaphragms. The research on shear walls has been oriented to low-height residential structures only and the configuration of the typical wall studied is shown in Figure 1-1. A typical wood frame shear wall consists of a 1200 or 2400 mm long frame structure, with 2x4” studs typically spaced at 400 mm on centres, double end studs, and 2x4” 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 (1976) obtained test data from a large number of shear wall tests with sheathing panels of various materials and formulated two empirical design methods. Richard, Daudeville, Prion, & Lam (2002) and Williamson & Yeh (2003) 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 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) (American Wood Council, 2015), 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, Waltz, & Skaggs (2008) to define the nominal unit shear capacities, among other properties, used for design.

To obtain a representative data set to support a design methodology, Salenikovich & Dolan (2003) 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 (2006) performed similar testing, but with fiberboard as sheathing material. Rosowsky, Elkins, & Carroll (2004) 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 & Fowler (1980) 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, Hirashima, Hatayoma, & Kanaya (1981) 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 (2003) 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, Prion, & He (1997) studied the lateral resistance of shear walls with nonstandard large OSB sheathing panels (2.4 x 7.3 m) and standard size OSB sheathing panels (1.2 x 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, Lam, & Prion (2001) tested twelve 2.4 x 2.4 m walls with large (2.4 x 2.4 m) and standard (1.2 x 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, Gupta, Rosowsky, & Dolan (2005) 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, Dean, & Shenton (2006), 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 SDPWS (American Wood Council, 2015) does not consider the favourable effects of vertical loads in the lateral response of shear walls.

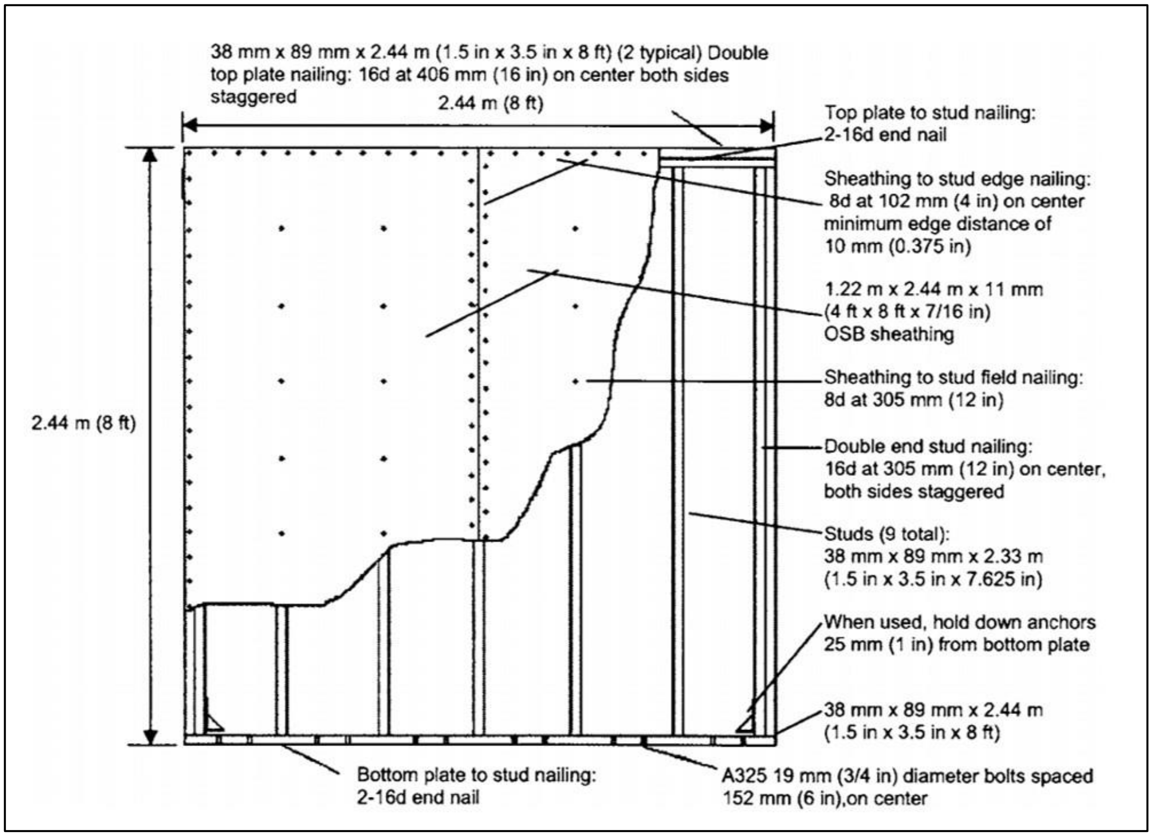


Figure 1-1 Typical shear wall specimen (Lebeda et al., 2005)

Nowadays middle-height timber buildings in the USA are designed using the current edition of the International Building Code (International Code Council, 2006), which requires to follow the design procedure indicated in ASCE/SEI 7-05 . Those provisions are collected in SDPWS (American Wood Council, 2015), where properties and requirements for wind and seismic loads are detailed for wood members, fasteners and assemblies. A performance-based design method for wood frame buildings was presented by Pang & Rosowsky (2010), where a direct displacement design procedure is used to carry out the design. The behaviour of the shear walls is characterized with non-linear modelling using CASHEW program (Folz & Filiatrault, 2002) 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. Therefore, the current code provisions and other new design methods do not consider the contribution of the rotational stiffness of the sturdy end studs and the strong hold-downs to the lateral behaviour of shear walls.

Mid-height timber buildings require a shear wall configuration that allows them to resist large vertical 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 Figure 1-1. Investigations on the lateral response of walls with sturdy end studs and strong hold-downs (typically needed in this type of buildings) are scarce. Sadeghi Marzaleh, Nerbano, Sebastiani Croce, & Steiger (2018) 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 evaluate de effects on the lateral response only of strong hold-downs and sturdy end studs. In this paper, to evaluate the behaviour of the walls three tests were executed and analytical models was generated and were concluded that the existing analytical methods underestimate the shear stiffness of the tested walls. In this investigation, there were constructional details that were highly influential on the shear strength (sheathing panels at the location of anchorages).

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 middle-height buildings, and to evaluate how well the current SDPWS (American Wood Council, 2015) 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 (Instituto Nacional de Normalización, 2009) 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 METHODS

The method used to determine mechanical properties of wood frame shear walls is the experimental test of wood frame shear walls and subsequent analysis of test data.

2.1 Test Specimens

The specimens with the configuration shown in Figure 2-1 were tested under in-plane shear loading. The walls were 2470 mm height with three different lengths: 1200, 2400 and 3600 mm. The structure of the walls consisted of MGP10 graded Chilean radiata pine elements: 2x6" vertical studs spaced at 407 mm, as well as 2x6" double plates at the top and bottom of the wall. The 2x6" elements are 36x138 mm. The sturdy end studs consist of five vertical studs (see Figure 2-1 and Figure 2-2). Double vertical studs were placed at the edges of 1200 by 2400 11.1-mm thick OSB panels mounted on both faces of the wall for ease of nailing the panels to the wood frame. Nails ϕ3x70 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 ϕ1x10" bolts to the sturdy end studs, and with one ϕ1-1/8x10" bolt to the foundation, as shown in Figure 2-2. To prevent sliding of the wall, ϕ1x10" 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 2-1. 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.

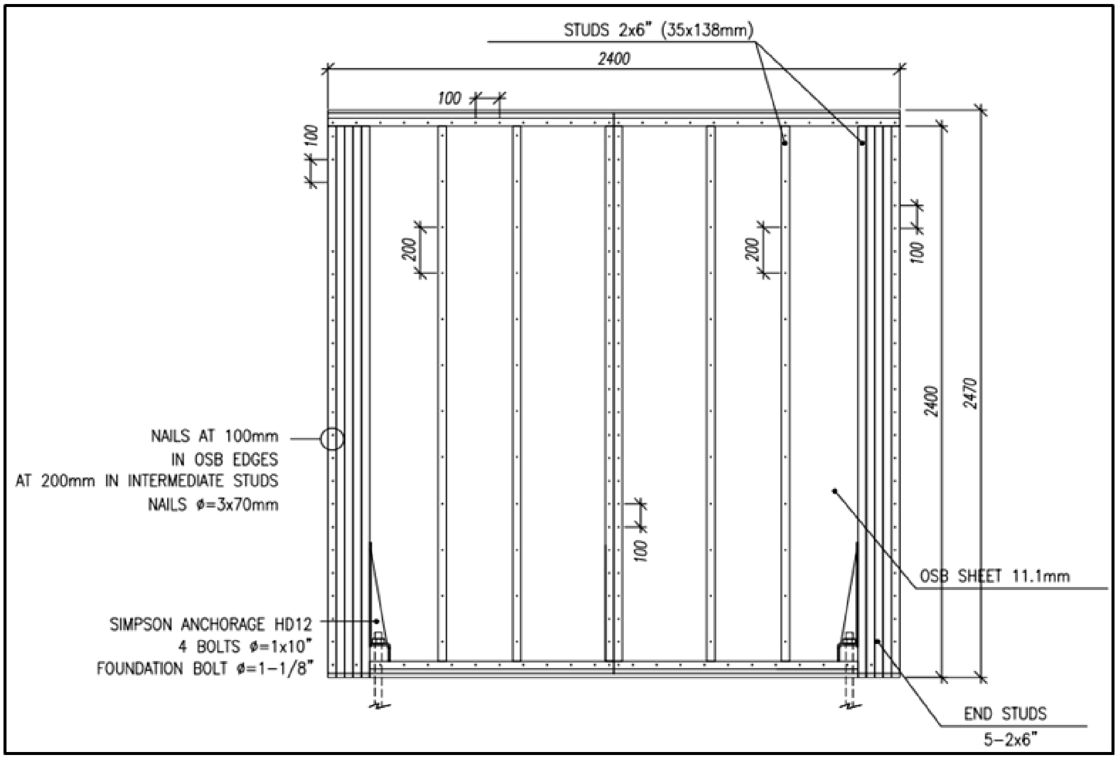


Figure 2-1. Configuration of a 2400 mm shear wall (lengths in millimetres).

Figure 2-2. Hold-down anchorage and sturdy end studs

Table 2-1. List of tested specimens

Notation	Protocol	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

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 displacement of the wall. 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 Figure 2-3.

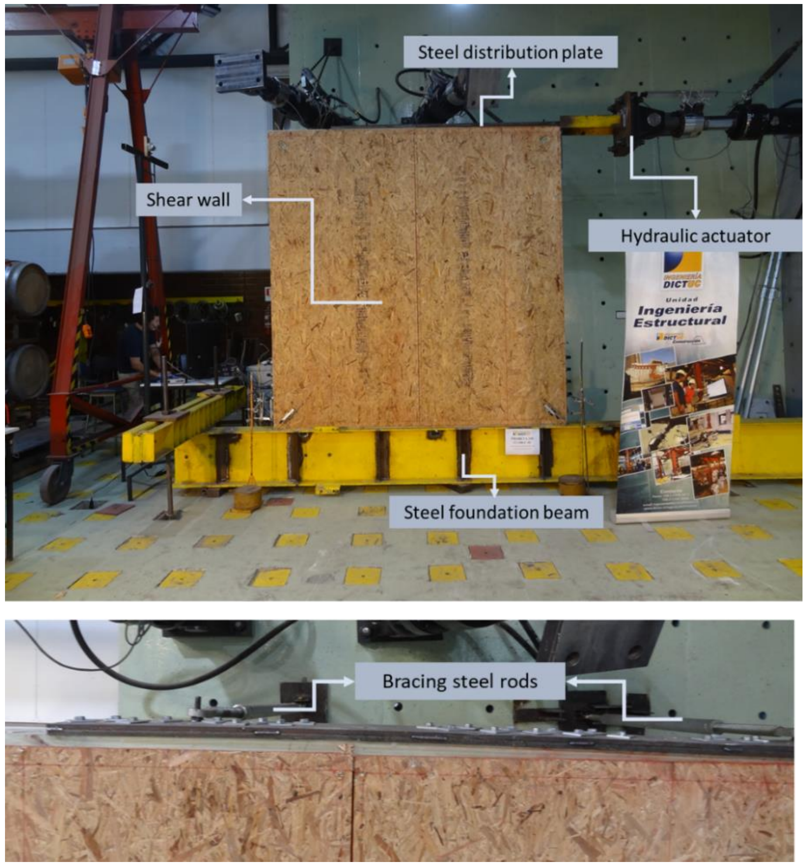


Figure 2-3. Test setup

2.3 Test procedure

The CUREE loading protocol (Krawinkler, Parisi, Ibarra, Ayoub, & Medina, 2001) 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. The cyclic loading history consisted of three types of cycles: initial cycles, which are executed at the beginning of the loading history; a primary cycle, which is a cycle larger than all of the preceding cycles; and trailing cycles that have an amplitude equal to 75% of the amplitude of the preceding primary cycle. Figure 2-4 shows the normalized displacement of the CUREE loading test protocol for cyclic tests. This protocol was applied in all cyclic tests, which were executed up to failure of the walls.

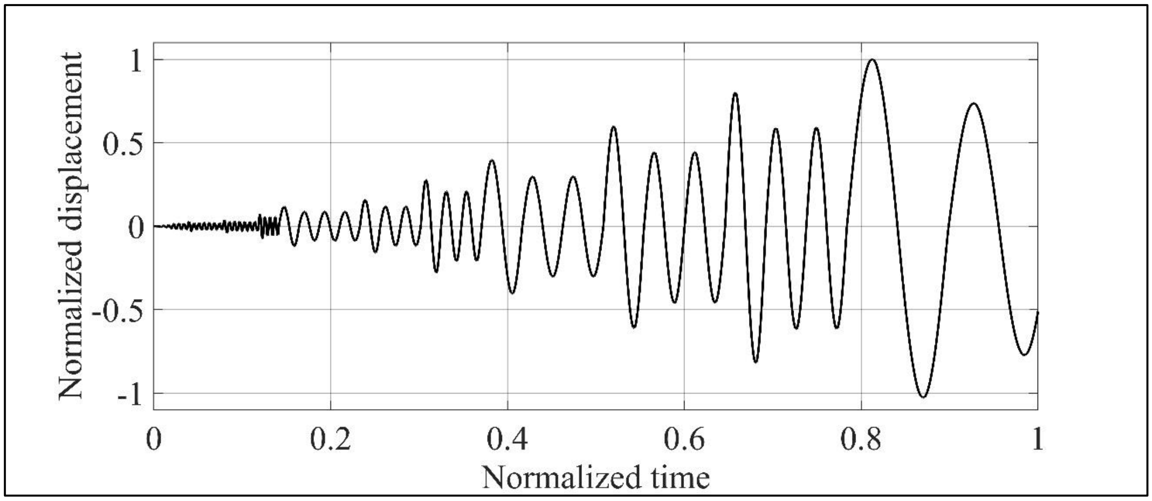


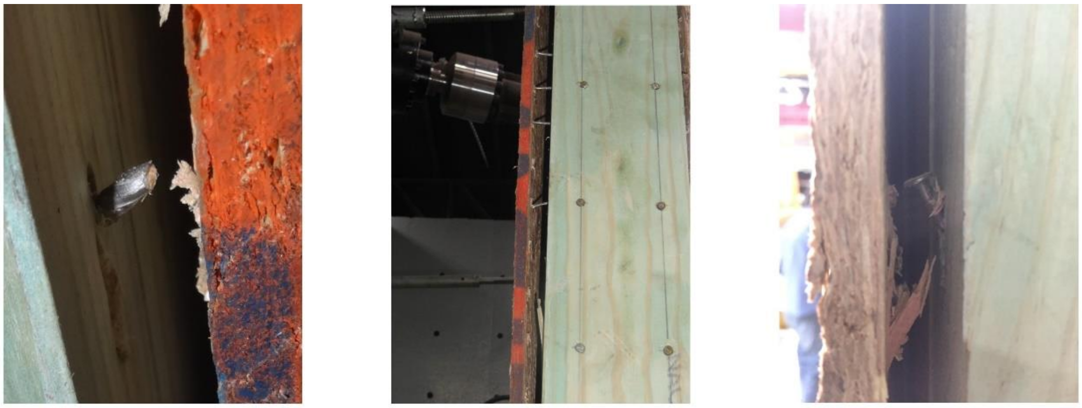
Figure 2-4. Loading history for cyclic load test.

3 RESULTS AND DISCUSION

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 (American Wood Council, 2015) provisions, to see how well the code expressions fit with the measured behaviour of the walls.

3.1 Failure mode

The walls presented the typical failure mode observed in most walls tested by authors (Johnston et al., 2006; Lebeda et al., 2005; Shenton III, Dinehart, & Elliott, 1998). The failure mode observed in all the walls was due to the cutting of sheathing nails (Figure 3-1a), pull out of the nails in the edges of the wall sheathing (Figure 3-1b) and crushing of OSB panel by nails head (Figure 3-1c), while the frame structure remained undamaged or slightly damaged. The failure of the walls commonly occurred by a combination of the three mentioned modes. Nevertheless, in two tests the frame structure of the walls experienced considerable damage. In the case of wall C120-05-02 the bottom plate broke in its entire length (Figure 3-2a); in wall M120-05-02 the zone of connection between the bottom plate and the end studs was extensively damaged (Figure 3-2b) and the end studs failed in tension (Figure 3-2c and Figure 3-2d) at very large lateral displacements.

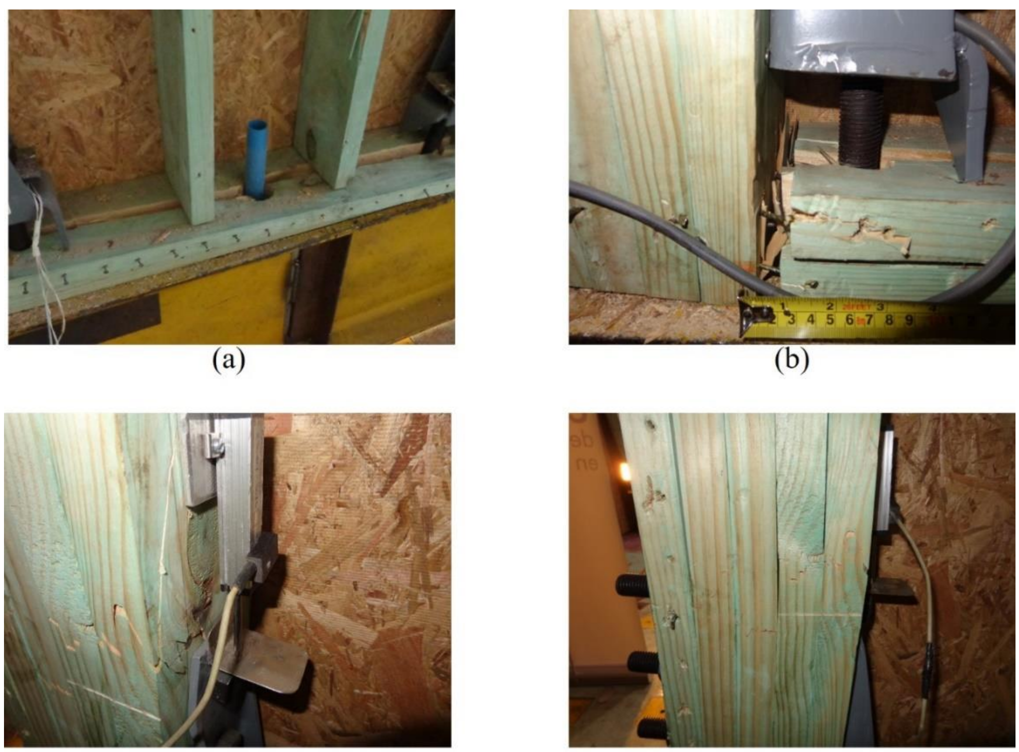


(a) Cutting of sheathing nails

(b) Pull out of the nails

(c) Crushing of OSB sheet by nails head

Figure 3-1. Failure mode of the walls



(a) (b) (c) (d)

Figure 3-2. Damage in bottom plate and end studs

3.2 Force-displacement response and main results

The hysteresis curves of five configurations of walls are shown in Figure 3-3. The shape of the curves of the walls is similar to those reported by other authors for typical wall configurations (Johnston et al., 2006; Lebeda et al., 2005; Shenton III et al., 1998). From these hysteretic response curves the envelope curves for positive and negative displacement loading were obtained. Initially, the response is approximately linear up to drift values of 0.5% to 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 rapidly.

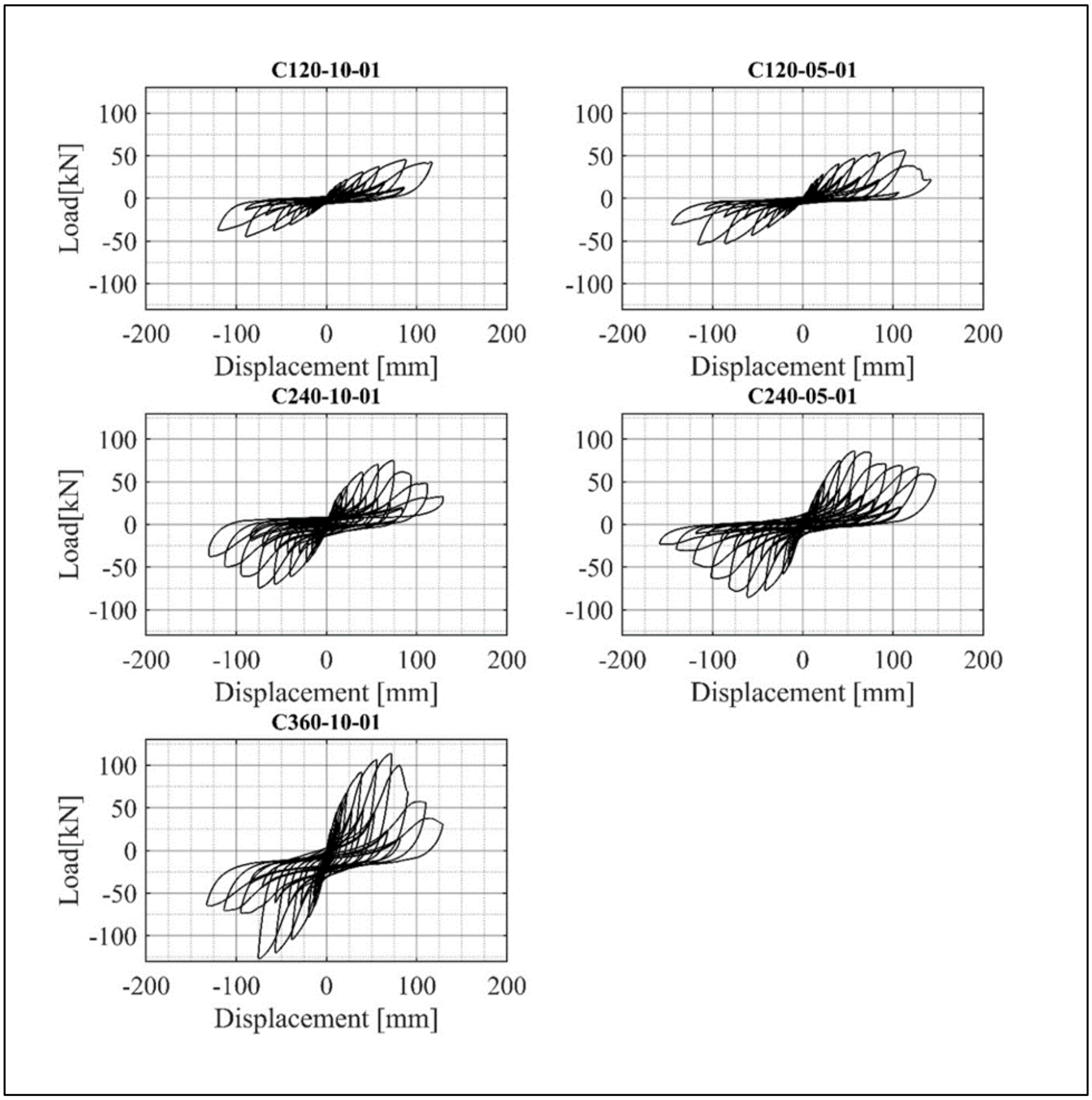


Figure 3-3. Typical hysteretic response for cyclic tests

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) are summarized in Table 3-1. P_{peak} is the maximum load of the average of the two envelope curves of each cyclic test.

Table 3-1. Summary of interest values of 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

From Table 3-1 and Figure 3-3 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 under cyclic load were 44.4 and 56.5 kN for 100 mm and 50 mm nail spacing, respectively, a 27% difference; for walls under monotonic loads the corresponding loads were larger, 48.7 and 67.6 kN, a 39% difference. For the 2400 mm long walls, the average maximum load of walls under cyclic loads were 76.3 and 89.2 kN, a difference of 17%; for the walls under monotonic loading the average maximum loads were 86.8 and 96.4

kN, 11% difference. 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 (2003), who concluded that cyclic loading reduced the peak load in average by 12%.

In Table 3-2 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%.

Table 3-2. Unit strength for different wall lengths (in kN/m)

	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	-

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. 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 Figure 3-4). The yield displacement was calculated from the equivalent energy elastic-plastic (EEEP) curve according to ASTM. E2126-11 (American Society of Civil Engineers, 2012). 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 Figure 3-4. The values of Δ_y and Δ_u are listed in Table 3-1.

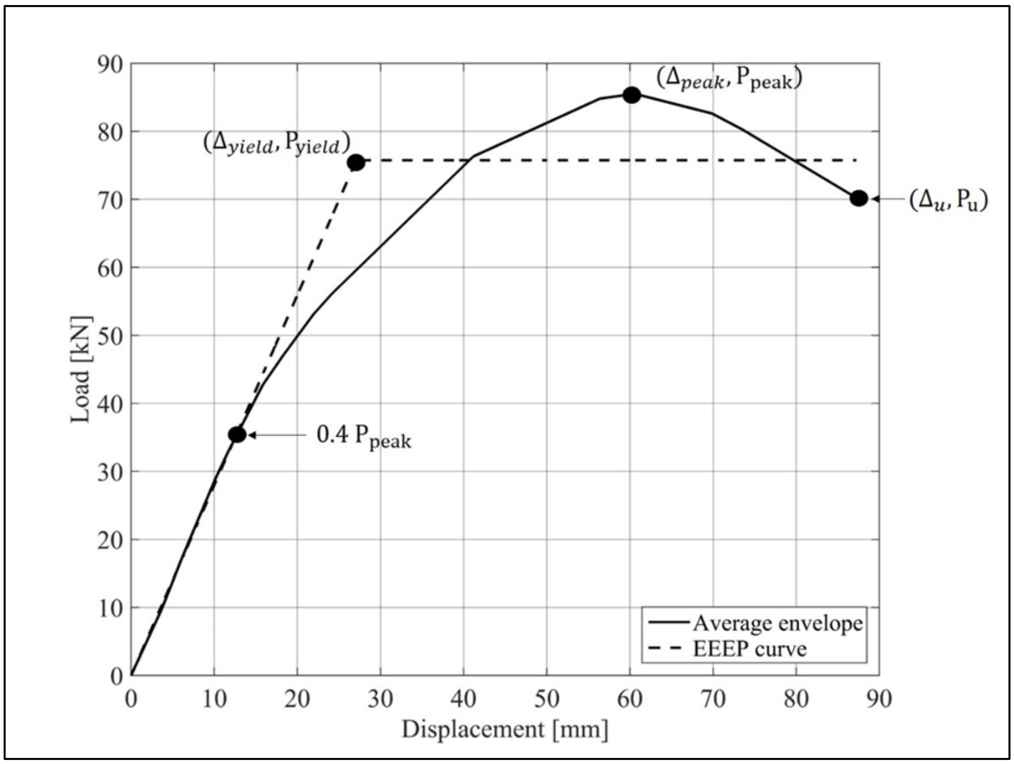


Figure 3-4. Average envelope and EEEP curve of wall C240-05-01

The yield displacement Δ_y varied between 22 and 40 mm (Table 3-1), which corresponds to a drift of approximately between 1.0 and 1.5%. 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 and 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, it is a parameter 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 depends only of the wall components. 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. Figure 3-5 illustrates a loop of a hysteretic curve and the parameters for calculating the damping ratio. EVD for cycle is calculated as follows:

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

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^-.

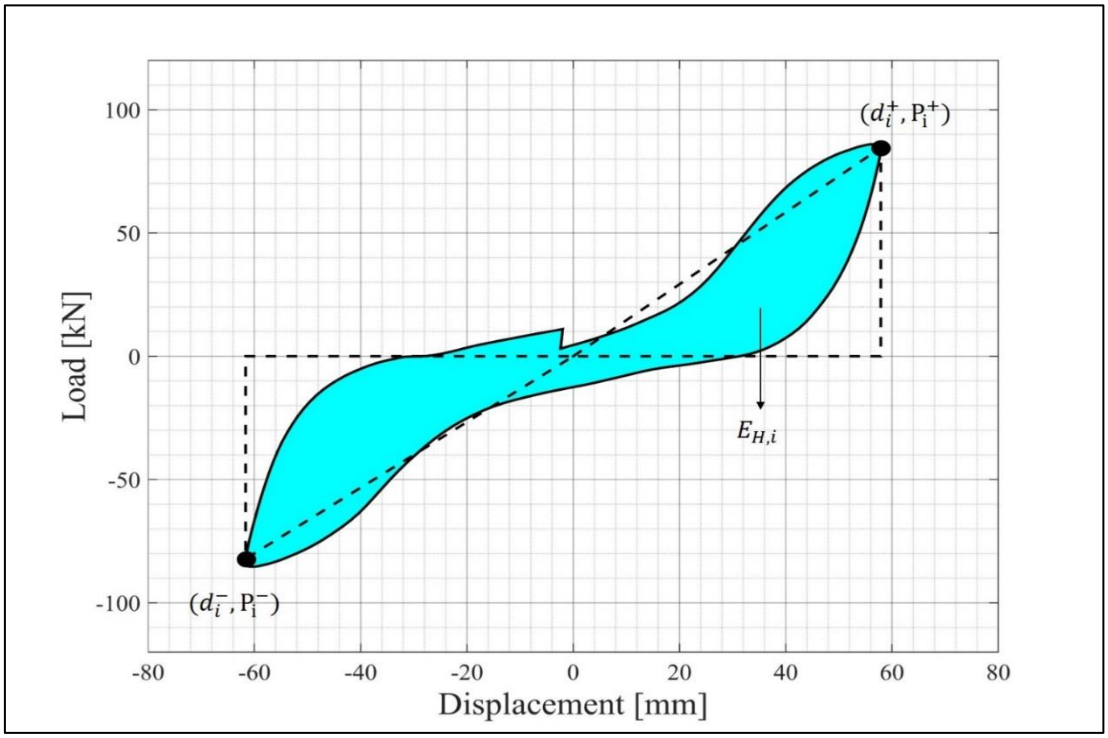


Figure 3-5. Illustration of a loop for damping calculation

The characteristic value of EVD (ξ) was calculated as the 10% percentile of EVD of all the cycles. In Figure 3-6 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 y 10%, with an average value of 9% and a standard deviation of 1%.

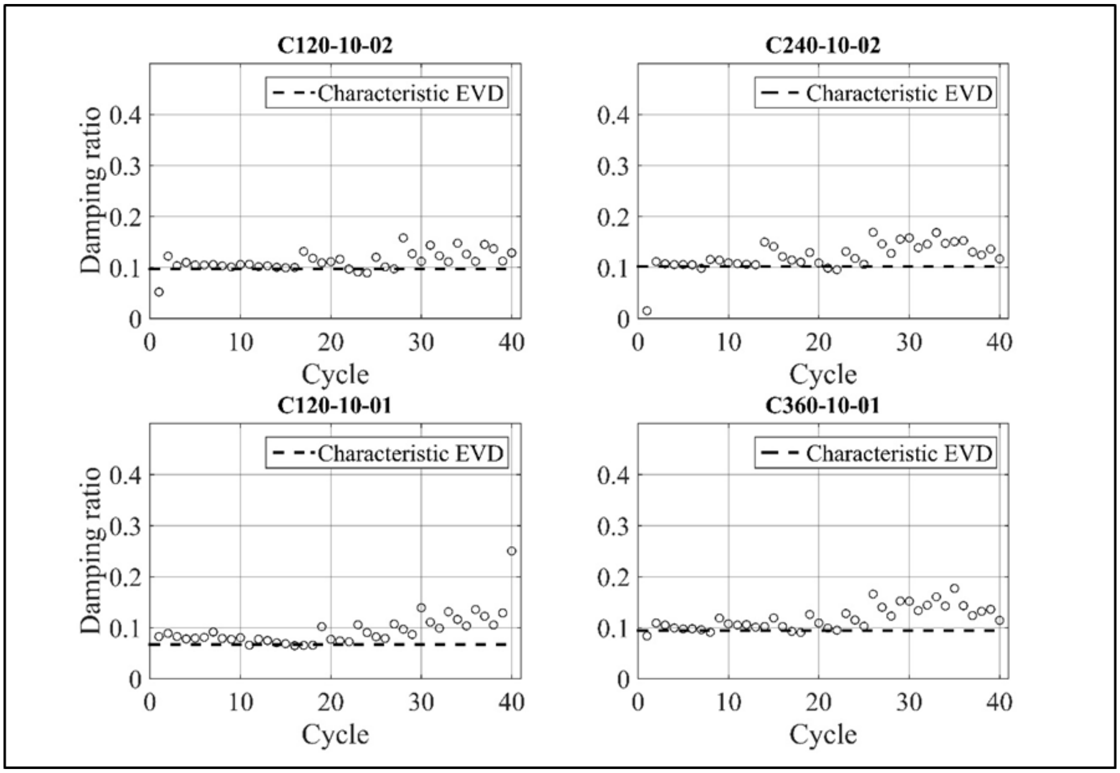


Figure 3-6. Equivalent viscous ratio for every loop of ten cyclic tests and characteristic EVD

3.6 Envelope response and stiffness

Figure 3-7 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 Table 3-3 and Table 3-4 are presented the measured unit shear strength ν_{exp}, the unit shear strength calculated using SDPWS (American Wood Council, 2015) 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 (American Wood Council, 2015) expresion 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 to 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.

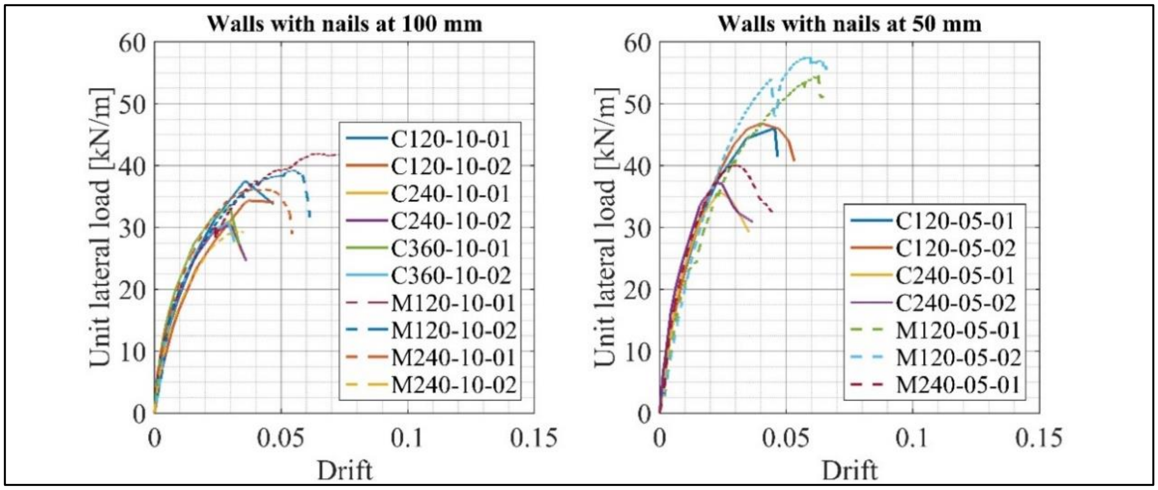


Figure 3-7. Unit lateral loads vs drift (Envelope curves).

Table 3-3. Unit load capacity and stiffness of walls with nails at 100 mm.

Specimen	v_{exp} (kN/m)	v_{SDPWS} (kN/m)	v_{exp}/v_{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 3-4. Unit load capacity and stiffness of walls with nails at 50 mm.

Specimen	v_{exp} (kN/m)	v_{SDPWS} (kN/m)	v_{exp}/v_{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**

The unit stiffness was calculated at 40% P_{Peak} of the load-displacement curves, which corresponds to a drift level of 0.4% to 0.8% in walls under cyclic loading and drift levels of 0.5 to 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 Figure 3-8.

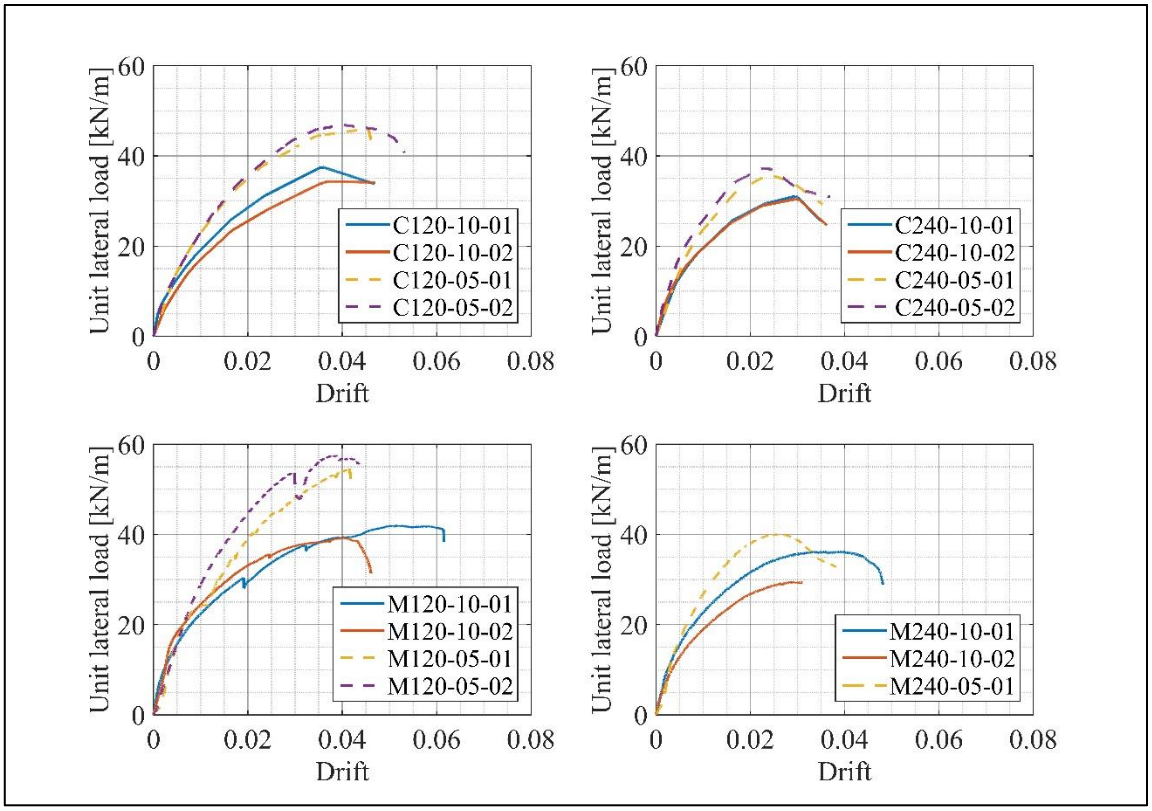


Figure 3-8. Effect of nail spacing on performance of the walls

In Figure 3-9 are shown the load displacement curves of walls with different lengths and nails spaced at 100 mm. It is possible to observe 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 tendency can be observed for the walls with nails spaced at 50 mm.

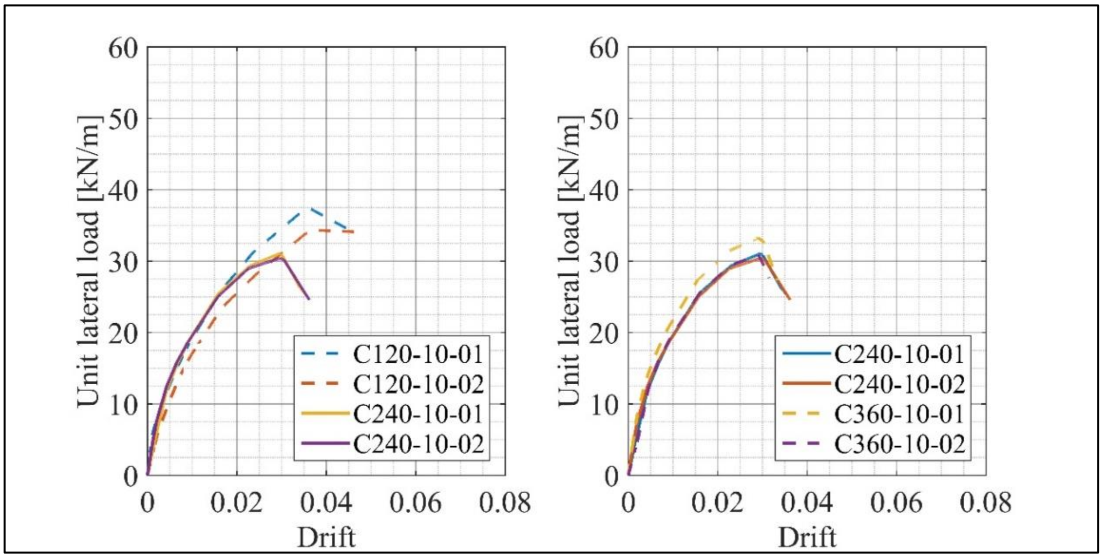


Figure 3-9. Effect of wall length on performance of the walls

4 DESIGN CONSIDERATIONS

A comparison between measured values in the tests and SDPWS provisions (American Wood Council, 2015) is presented in the current section. Strength, stiffness and 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 define if provisions are appropriate to design wood frame shear walls with sturdy end studs and strong hold-downs, or maybe it is necessary to propose new provisions for this kind of walls.

4.1 Design values

In Table 3-3 and Table 3-4 are presented the measured unit shear strength ν_{exp}, the unit shear strength calculated using SDPWS (American Wood Council, 2015) ν_{SDPWS}, the ratio of calculated to measured strengths ν_{exp} to ν_{SDPWS}, the unit stiffness at 40% P_{peak} (K_{40}), the unit stiffness K_{SDPWS} calculated using the SDPWS expresion (American Wood Council, 2015) for deflection (eq. 2) and the ratio of K_{40} to K_{SDPWS}.

4.1.1 Strength

The values of the calculated unit shear strength (ν_{SPDWS}) were obtained from Table 4.3A of the SDPWS (American Wood Council, 2015). 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 4-1 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 (Johnston et al., 2006; Lebeda et al., 2005; Line et al., 2008; Shenton III et al., 1998). The walls are 2.4 x 2.4 m shear walls (Figure 1-1), 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.

Table 4-1. Average values of unit strength of typical tested shear walls

Test	Strength (kN/m)	SPDWS Strength (kN/m)	Strength ratio
Line et al. (2008)	13.5	10.2	**1.32**
Lebeda et al. (2005)	13.3	10.2	**1.30**
Johnston et al. (2006)	14.7	10.2	**1.44**
Shenton III et al. (1998)	11.7	10.2	**1.15**
Average	**13.2**	**10.2**	**1.30**

4.1.2 Stiffness and deflections

The stiffness of the walls can be calculated from SDPWS equation 4.3-1 (American Wood Council, 2015), used to calculate by elastic analysis the lateral displacement. The horizontal inter-story displacement δ_{ws} used to design, is calculated as (eq. 2):

$$\delta_{ws} = \frac{8\,\nu h^3}{EAb} + \frac{\nu h}{1000\,n\,G_a} + \frac{h\,\Delta_a}{b} \tag{2}$$

where:

ν = unit lateral load (kN/mm)

h = wall height = 2470 mm

E = modulus of elasticity of the wood from Table 4.b Nch1198 = 10.000 MPa (Instituto Nacional de Normalización, 2014)

A = end studs area = 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 = $\begin{cases} 7.355\ (\text{nails at } 50\ \text{mm}) \\ 3.853\ (\text{nails at } 100\ \text{mm}) \end{cases} \frac{\text{kN}}{\text{mm}}$

Δ_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 Figure 4-1, while K_{hd} is obtained from Simpson StrongTie C-C-2017 catalogue (Simpson StrongTie, 2017) and reached a value of 13.7 kN/mm for the used hold-down. Equations 3 through 6 show the calculations of the unit stiffness of the walls K_{SDPWS}.

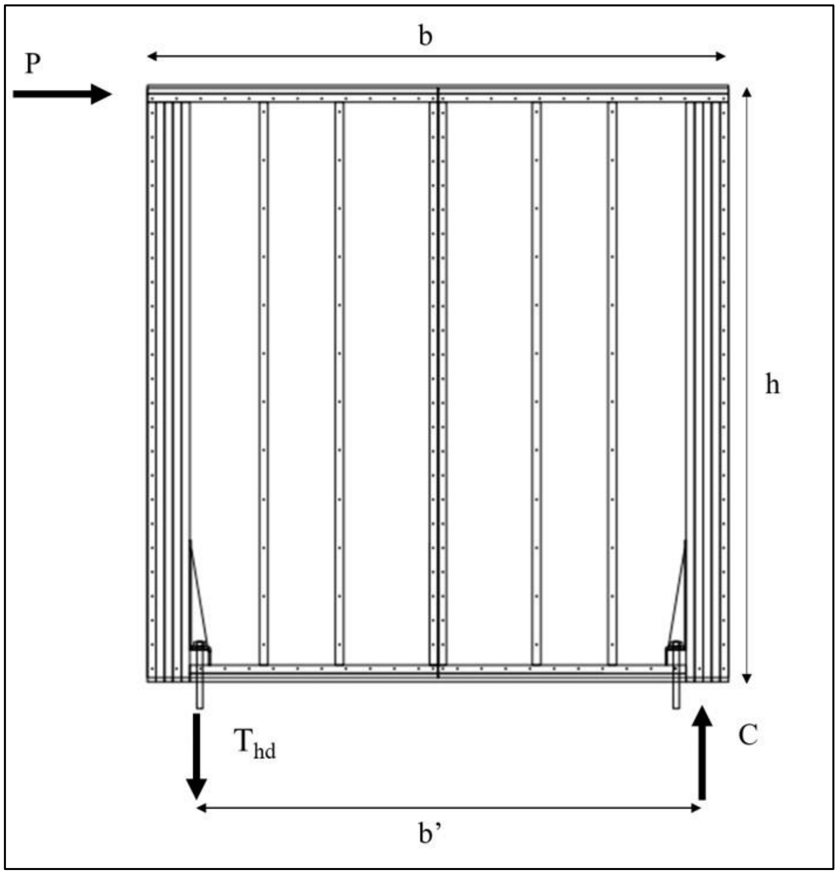


Figure 4-1. Diagram of the overturning forces on the wall.

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

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

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

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

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

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 Figure 4-2, 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 stiffnesses were similar.

The unit stiffness measured at 40% P_{Peak} are smaller than the stiffness calculated from equation 4.3-1 of SDPWS (American Wood Council, 2015), 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 Table 3-3 and Table 3-4).

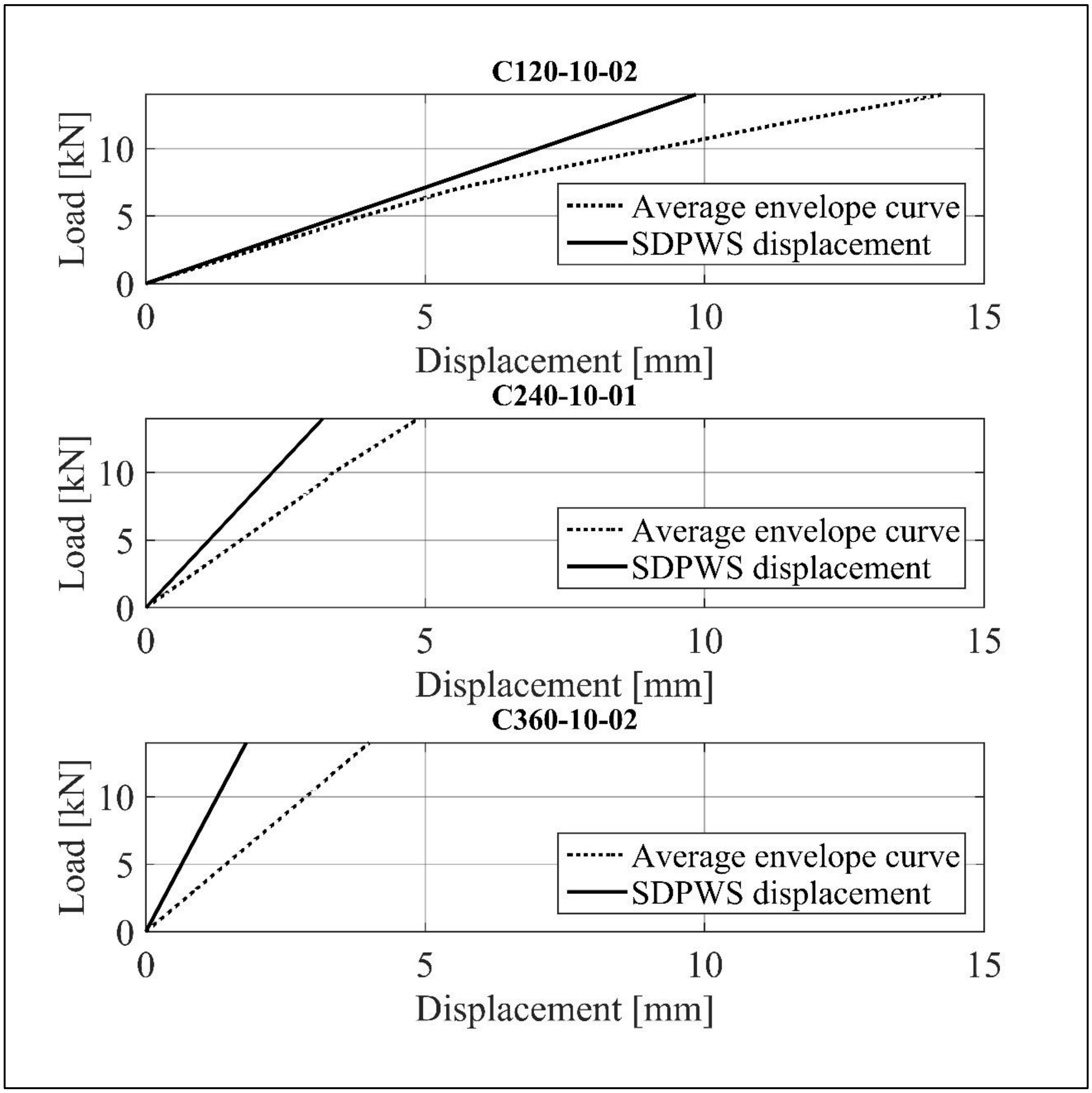


Figure 4-2. Measured and calculated load-displacement curves of walls with nails at 100 mm.

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 Figure 3-4. 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 (American Wood Council, 2015) (see Figure 4-3):

$$G_a = \frac{P}{\delta_{shear}} \frac{h}{n\,L} \rightarrow \boldsymbol{G_a} = \boldsymbol{K_{shear}} \frac{\boldsymbol{h}}{\boldsymbol{n\,L}} \quad (7)$$

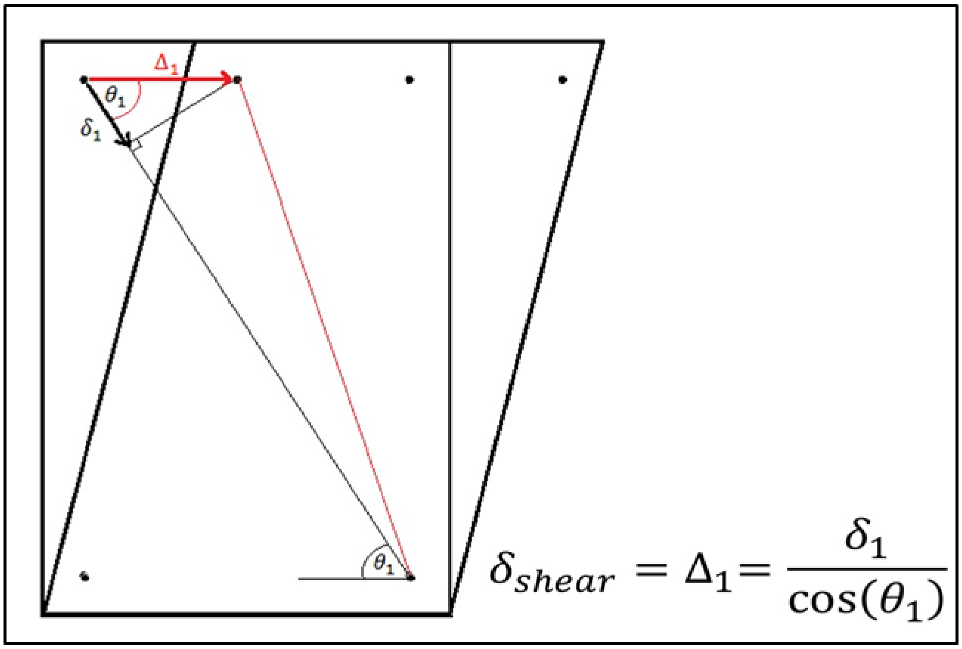


Figure 4-3. Shear component of wall deflection

This value of G_a is compared with the value of G_a provided by Table 4.3A of the SDPWS (American Wood Council, 2015) and the difference are observed in the Table 4-2 and Table 4-3. It is observed that measured G_a values are in average 40% to 50% smaller than the values obtained from Table 4.3A of the SDPWS (American Wood Council, 2015). 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, probably due to problems with the transducers used to measure diagonal deformations.

Table 4-2. Apparent shear stiffness comparison of walls with nails at 100 mm.

Wall (nails at 100 mm)	Ga [kN/mm] SDPWS	Ga [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**

Table 4-3. Apparent shear stiffness comparison of walls with nails at 50 mm.

Wall (nails at 50 mm)	Ga [kN/mm] SDPWS	Ga [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**

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 (American Wood Council, 2015) are too large, as it is observed in Table 4-2 and Table 4-3. In Table 3-3 and Table 3-4 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 Table 3-3 and Table 3-4 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 (American Wood Council, 2015) 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 (Instituto Nacional de Normalización, 2009) 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 4-4 and Table 4-5, shows the measured loads at different drift values (0.2%, 0.3% and 0.5%), taken from the monotonic responses and the measured envelope curves. It is observed that for drifts of 0.2%, 0.3%, 0.4% and 0.5%, the load in walls with nails at 100 mm are approximately 23%, 32%, 38% and 43% of the corresponding maximum load, respectively. For walls with nails at 50 mm, the corresponding average values of load are 16%, 25%, and 40%, respectively. In the last case, the variability of those values is large, but the loads less than 35% P_{Peak} for the drift of 0.3% and less than 50% P_{Peak} for the drift of 0.5%. At load levels less than 40% of the peak loads, the tested walls did not show any damage. This suggests that a drift limit larger than 0.2% may be used in the Chilean Seismic design code for mid-height timber buildings.

Table 4-4. Load levels to different drifts values of walls with nails at 100 mm.

Wall	**Maximum load [kN]**	Load at drift 0.2% [kN]		Load at drift 0.3% [kN]		Load at drift 0.4% [kN]		Load at drift 0.5% [kN]	
M120-10-01	**50.4**	11.2	22.2%	14.2	28.1%	16.6	33.0%	18.8	37.3%
M120-10-02	**47.1**	9.4	20.0%	16.0	33.9%	19.7	41.9%	21.9	46.5%
C120-10-01	**45.3**	10.4	22.9%	13.6	29.9%	15.9	35.1%	18.1	40.0%
C120-10-02	**43.5**	9.2	21.1%	12.5	28.6%	15.3	35.1%	17.7	40.7%
M240-10-01	**86.8**	20.9	24.0%	27.3	31.4%	32.7	37.7%	37.4	43.1%
M240-10-02	**70.5**	16.5	23.5%	22.4	31.8%	26.7	37.9%	30.7	43.6%
C240-10-01	**75.0**	18.8	22.3%	27.5	32.5%	29.9	39.8%	36.5	43.2%
C240-10-02	**77.5**	20.0	25.9%	26.3	34.0%	31.5	40.7%	35.3	45.5%
C360-10-01	**127.1**	31.5	24.8%	41.5	32.7%	50.1	39.4%	56.8	44.7%
C360-10-02	**114.6**	18.2	15.9%	29.6	25.8%	41.2	36.0%	48.8	42.6%
Average			**23.3%**		**31.9%**		**37.7%**		**43.6%**
Standard dev			**3.6%**		**3.4%**		**2.8%**		**3.1%**
CV			**0.15**		**0.11**		**0.08**		**0.07**

Table 4-5. Load levels to different drifts values of walls with nails at 50 mm.

Wall	**Maximum load [kN]**	Load at drift 0.2% [kN]		Load at drift 0.3% [kN]		Load at drift 0.4% [kN]		Load at drift 0.5% [kN]	
M120-05-01	**66.0**	4.7	7.1%	10.4	15.8%	16.0	24.2%	20.0	30.3%
M120-05-02	**69.2**	7.2	10.3%	11.0	16.0%	15.0	21.7%	19.0	27.5%
C120-05-01	**56.3**	10.8	19.1%	16.0	28.4%	19.3	34.3%	21.9	38.8%
C120-05-02	**56.8**	11.8	20.9%	16.0	28.2%	19.3	33.9%	22.5	39.6%
M240-05-01	**96.4**	13.3	13.8%	26.0	27.0%	34.1	35.4%	39.5	41.0%
C240-05-01	**86.0**	15.9	18.5%	24.9	29.0%	33.1	38.5%	39.9	46.3%
C240-05-02	**92.5**	21.5	23.2%	30.9	33.4%	40.1	43.3%	45.7	49.4%
Average			**16.1%**		**25.4%**		**33.0%**		**39.0%**
Standard dev			**5.9%**		**6.8%**		**7.7%**		**7.9%**
CV			**0.36**		**0.27**		**0.23**		**0.20**

A similar analysis was realized from four 2000 mm height reinforced concrete walls with different longitudinal and transverse reinforcing steel ratios. The obtained results were independent of the reinforcing steel ratios and are presented in Table 4-6.

Table 4-6. Load levels to different drifts values of concrete walls.

	Load at drift 0.2%	Load at drift 0.3%	Load at drift 0.4%	Load at drift 0.5%
Average level	41.2%	49.7%	61.2%	72.7%
standard dev.	2.3%	3.3%	4.0%	4.9%

At drift level of 0.5% of wood frame shear wall, reached load level is similar to the load level 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, then a limit drift level of 0.2% used in current limit of the Chilean Seismic Design Code is too conservative for wood frame walls studied in this paper.

It is necessary to consider new design limits for multi-storey timber structures. The walls oriented to mid-height timber buildings have not the same behaviour as the residential walls, thus, SDPWS (American Wood Council, 2015) provisions are not accurate to design the kind walls tested in this paper.

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 oriented 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 in most 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 strength and ultimate displacement than 2400 mm walls; however, there were not 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 calculated at 40% P_{peak} was similar for both values 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 (American Wood Council, 2015), 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 wood frame large scale structures, at least wood frame shear walls with sturdy end studs and strong hold-downs.

- Apparent shear stiffness (G_a) provided by SDPWS (American Wood Council, 2015) is the principal cause of the overestimation of lateral displacements. The measured value of G_a was lower than theoretical value of apparent shear stiffness and this is the main cause of the difference between theoretical and experimental deflections of the walls.

- 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% corresponds to undamaged response of wood frame shear walls.

6 ACKNOWLEDGEMENTS

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APPENDICES

APPENDIX A: INSTRUMENTATION OF THE WALL AND IMPORTANT DISTANCES

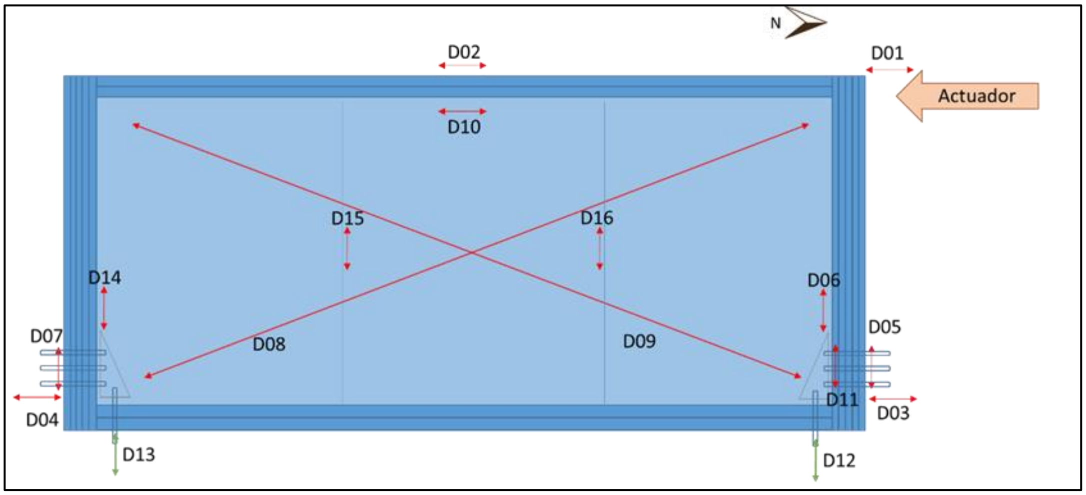

Figure A. 1 Instrumentation of the wall.

The measurements of the displacements were by means of transducers in positions indicate in the Figure A. 1. Each channel is associated to a column of the excel sheet of the output. The channel corresponding to each column is indicated in the Table A. 1.

Table A. 1. Excel sheet columns of the corresponding measured channel.

Excel column	Variables
1	Time
2	Load
3	D01
4	D02
5	D03
6	D04
7	D05
8	D06
9	D07
10	D08
11	D09
12	D10
13	D11
14	D12
15	D13
16	D14
17	D15
18	D16

Some important distances are:

Location of hold-down: The bolt that connect the wall with the foundation is located at 230 mm of exterior edge of the wall.

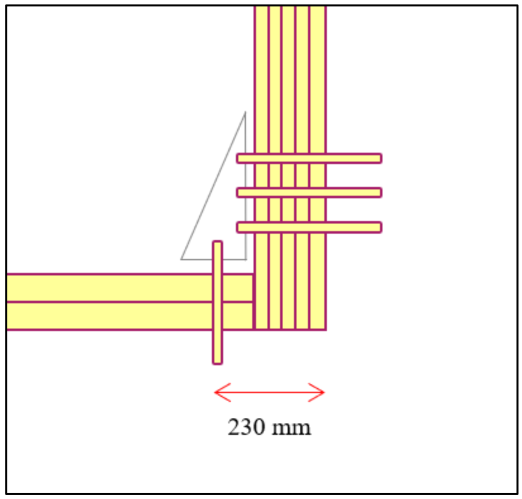

Figure A. 2. Hold-down location.

Location of diagonal transducers: A diagonal transducer measure the shear deformation as the displacement between two points located in the corners of the wall. Figure A. 3 shows the required distances to do the calculations in relation to shear behavior.

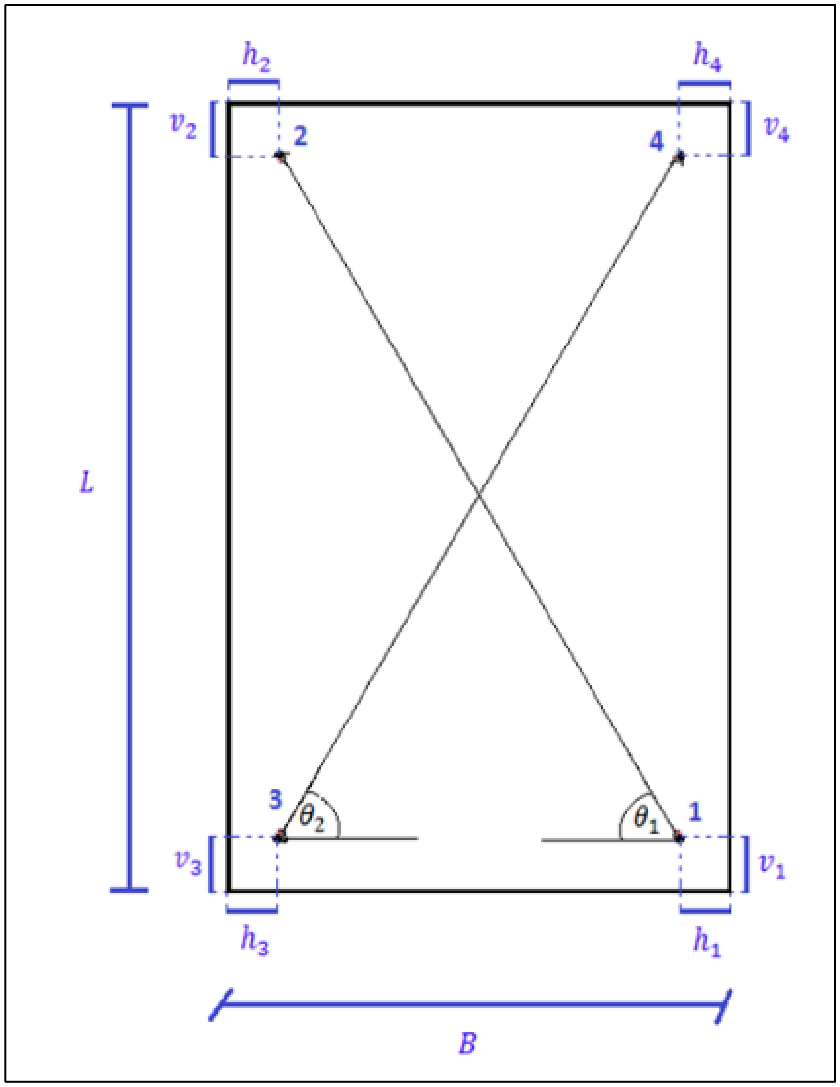

Figure A. 3. Location of the points used to measure the diagonal deformation.

Table A. 2. Loacation of diagonal transducers

Wall	h1	v1	h2	v2	h3	v3	h4	v4
M120-10-01	130	150	130	150	130	150	130	150
M120-10-02	130	150	130	150	130	150	130	150
M120-05-01	130	150	130	150	130	150	130	150
M120-05-02	130	150	130	150	130	150	130	150
M240-10-01	130	130	130	130	130	130	130	130
M240-10-02	130	130	130	130	130	130	130	130
M240-05-01	130	130	130	130	130	130	130	130
C120-10-01	120	130	120	130	120	130	120	130
C120-10-02	120	130	120	130	120	130	120	130
C120-05-01	120	130	120	130	120	130	120	130
C120-05-02	120	130	120	130	120	130	120	130
C240-10-01	130	130	130	130	140	130	130	130
C240-10-02	135	125	120	130	125	125	130	120
C240-05-01	130	125	130	145	130	130	125	150
C240-05-02	130	130	130	130	130	140	130	130
C360-10-01	160	115	150	110	155	110	155	130
C360-10-02	150	110	150	125	155	110	150	125

APPENDIX B: TESTS FOR MOISTURE CONTENT

Moisture content was calculated according ASTM D4441.

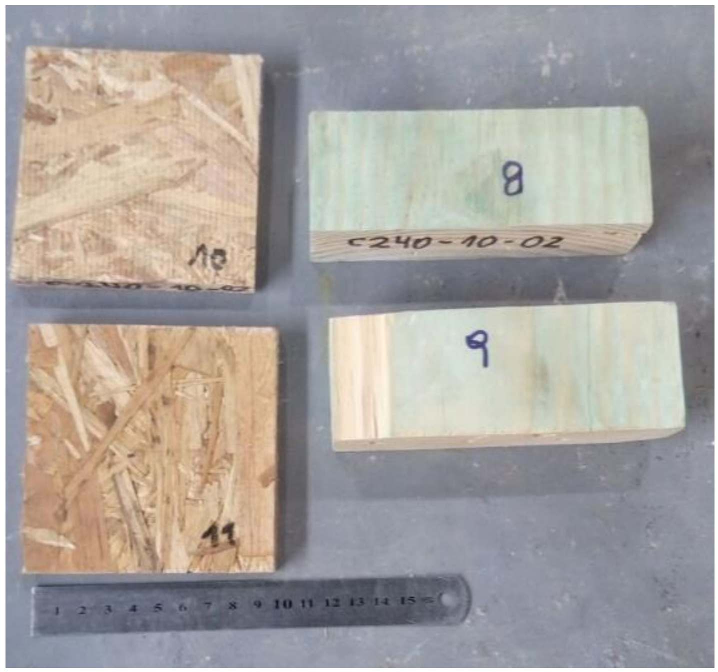

Figure B. 1. Specimens for moisture tests.

Table B. 1. Moisture content of vertical studs.

n°	Wall	Moisture
1	C240-05-01	9,4%
2	C240-05-01	9,4%
3	C240-05-01	9,6%
4	C240-05-01	9,5%
8	C240-10-02	9,3%
9	C240-10-02	9,3%
12	C240-05-02	9,7%
13	C240-05-02	9,7%
	Average	**9,5%**
	Standard dev.	**0,2%**
	CV	**1,7%**

Table B. 2 Moisture content of OSB panels.

n°	Wall	Moisture
5	C240-05-01	7,4%
6	C240-05-01	6,5%
7	C240-10-02	7,0%
10	C240-10-02	6,6%
11	C240-05-02	6,6%
14	C240-05-02	6,5%
15	C240-05-02	6,1%
	Average	**6,7%**
	Standard dev.	**0,4%**
	CV	**5,3%**

APPENDIX C: MECHANICAL PROPERTIES

Mechanical properties were calculated according NCh3028/1

Table C. 1. Results of bending tests (36 x 138 mm specimens).

Cód. Probeta	Ancho promedio [mm]	Espesor promedio [mm]	Contenido de humedad promedio [%]	Luz ensayo [mm]	Fuerza última [N]	Tiempo [s]	Módulo de elasticidad [Mpa]
FE-P001	138,3	35,7	13,0	2.054	19.485	0:44	13.213
FE-P002	138,6	35,7	12,1	2.054	11.483	0:22	9.411
FE-P003	139,1	35,5	11,7	2.054	16.589	0:40	12.920
FE-P004	138,9	35,9	12,5	2.054	15.261	0:39	12.387
FE-P005	139,1	35,8	13,1	2.054	19.138	0:44	13.088
FE-P006	138,5	35,6	11,9	2.054	11.506	0:20	10.195
FE-P007	138,4	35,7	11,9	2.054	21.864	0:46	12.486
FE-P008	138,9	36,2	11,9	2.054	19.927	0:45	12.955
FE-P009	138,9	35,6	14,2	2.054	14.131	0:40	7.792
FE-P010	139,1	35,7	12,2	2.054	14.125	0:25	8.446
FE-P011	137,6	35,4	13,0	2.054	17.579	0:47	14.170
FE-P012	139,4	35,7	12,7	2.054	8.359	0:27	10.530
FE-P013	139,3	35,6	12,0	2.054	13.026	0:47	11.126
FE-P014	138,6	35,7	12,4	2.054	17.228	0:42	11.579
FE-P015	138,6	35,7	11,3	2.054	9.732	0:24	10.698

APPENDIX D: LOAD-DISPLACEMENT CURVES

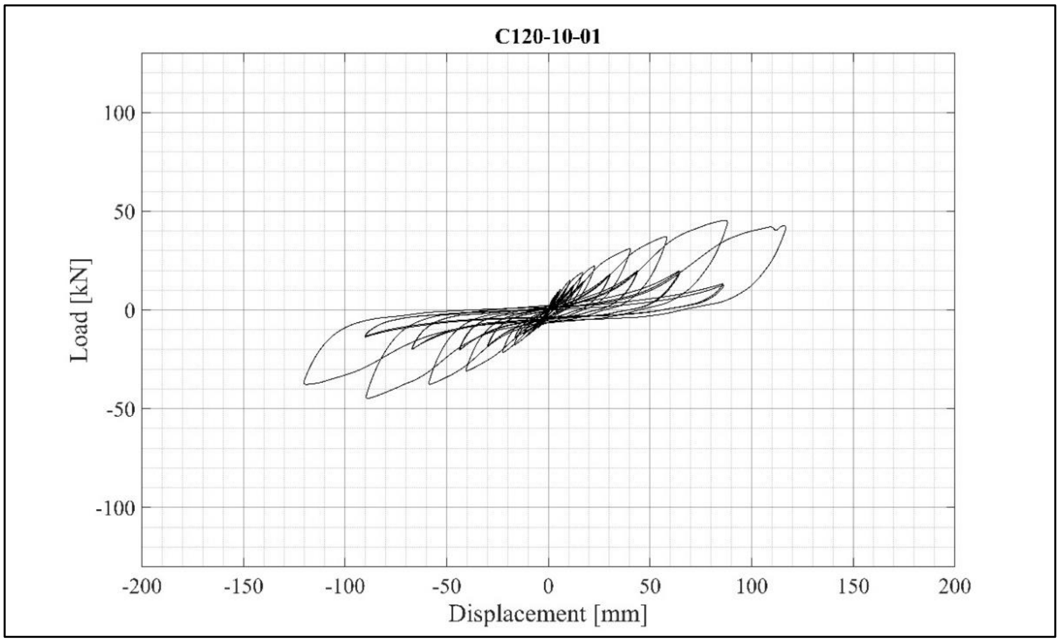

Figure D. 1. Load displacement curve of wall C120-10-01.

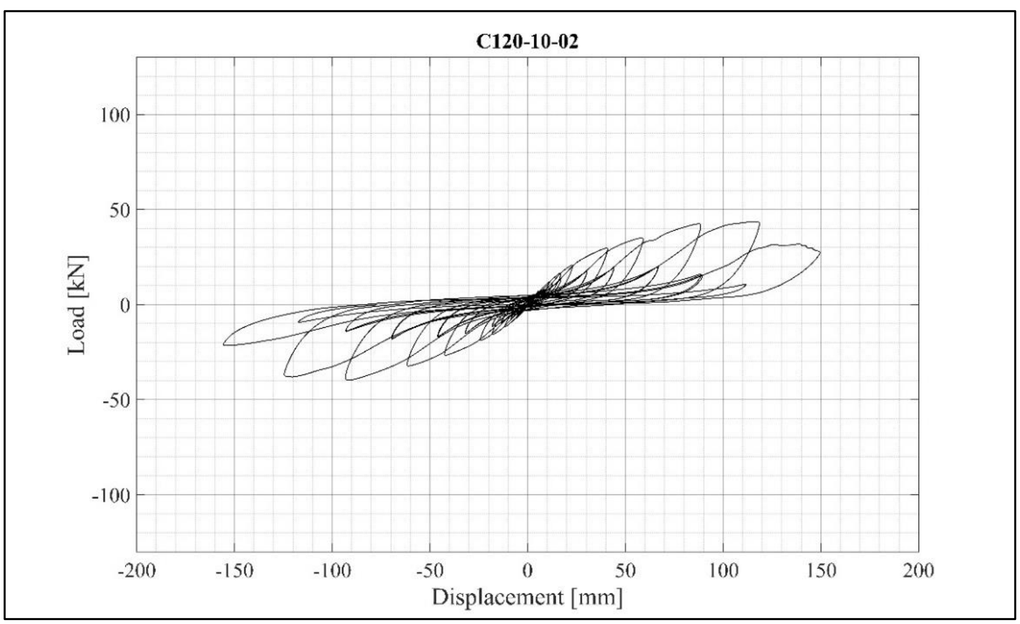

Figure D. 2. Load displacement curve of wall C120-10-02.

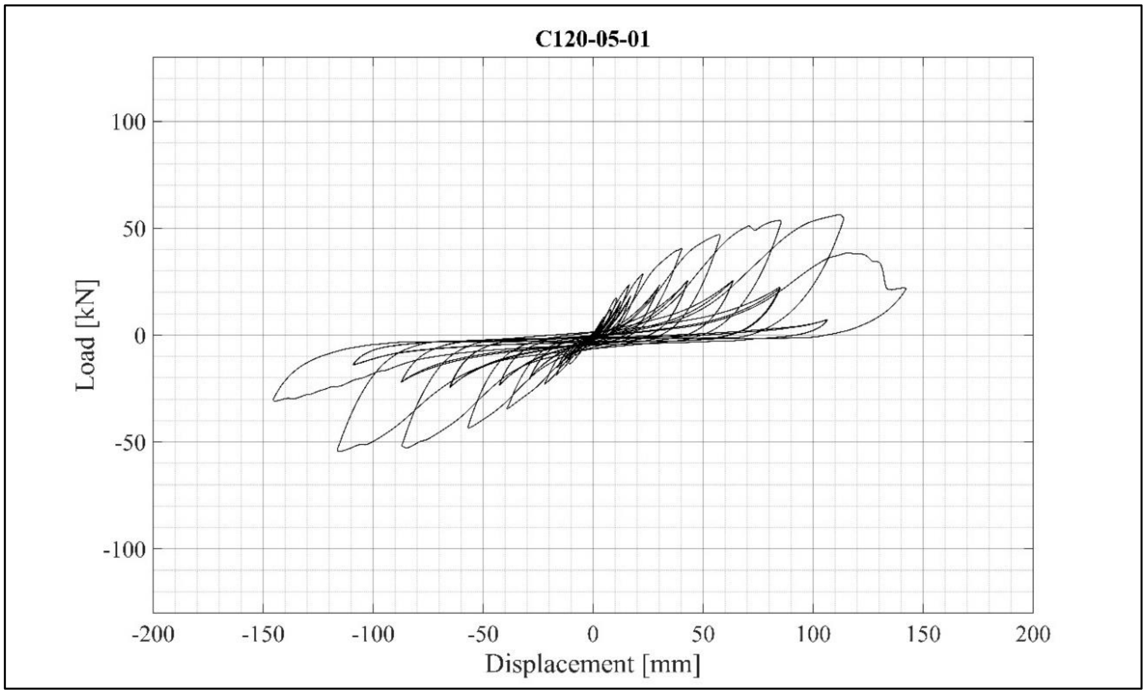

Figure D. 3. Load displacement curve of wall C120-05-01.

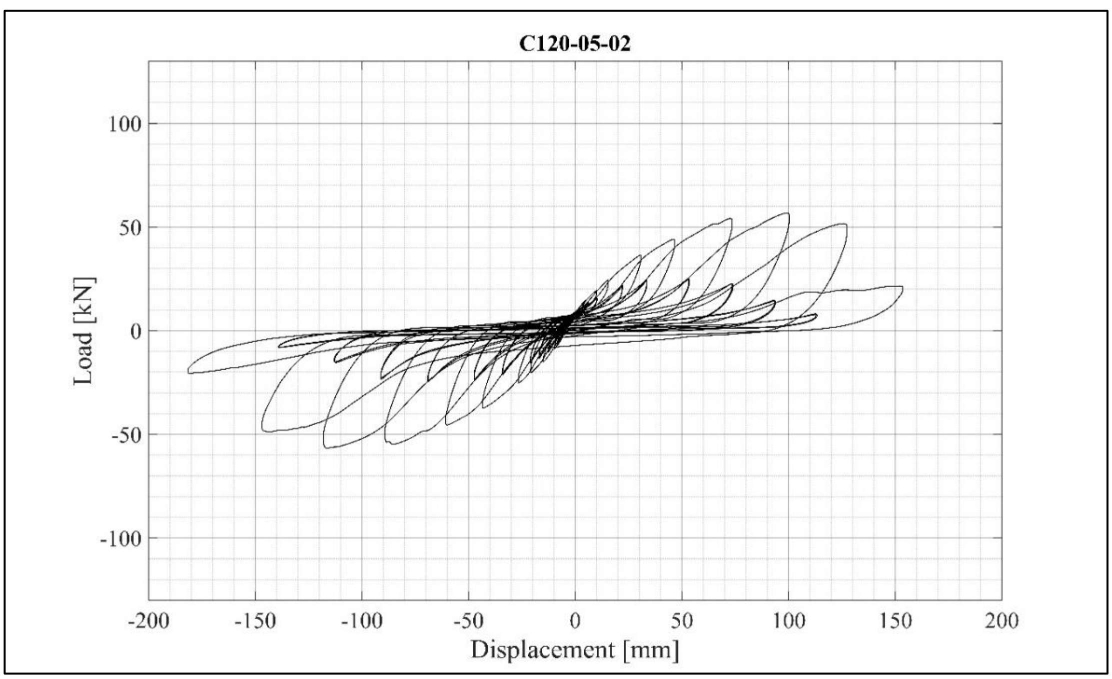

Figure D. 4. Load displacement curve of wall C120-05-02.

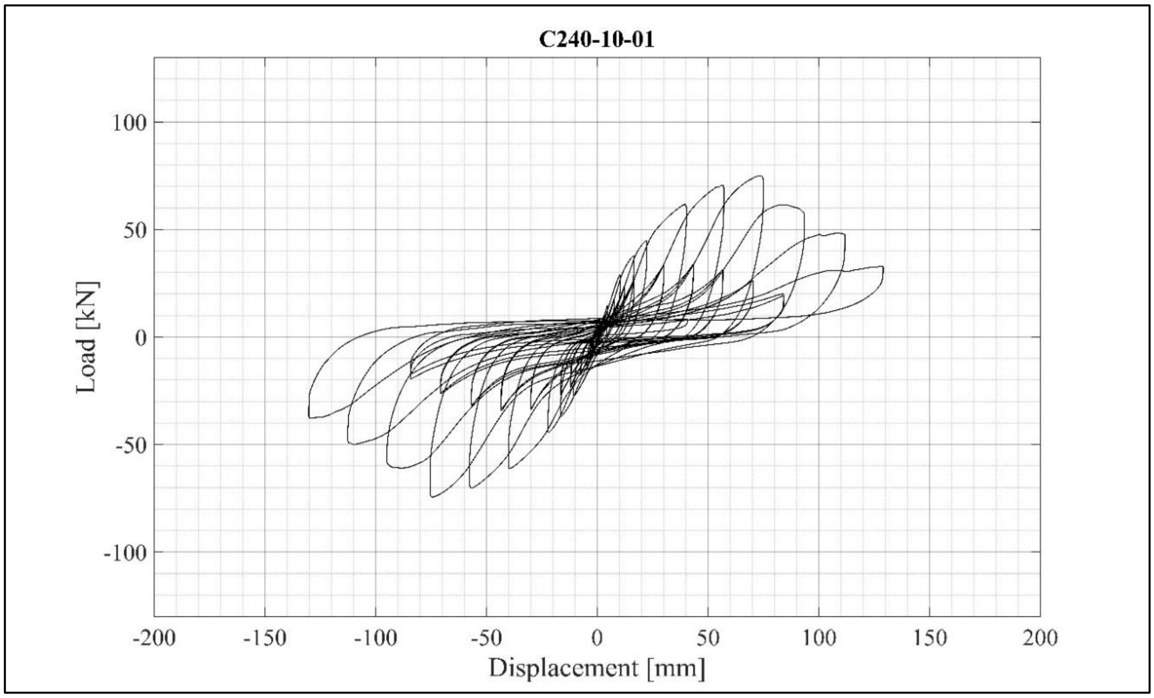

Figure D. 5. Load displacement curve of wall C240-10-01.

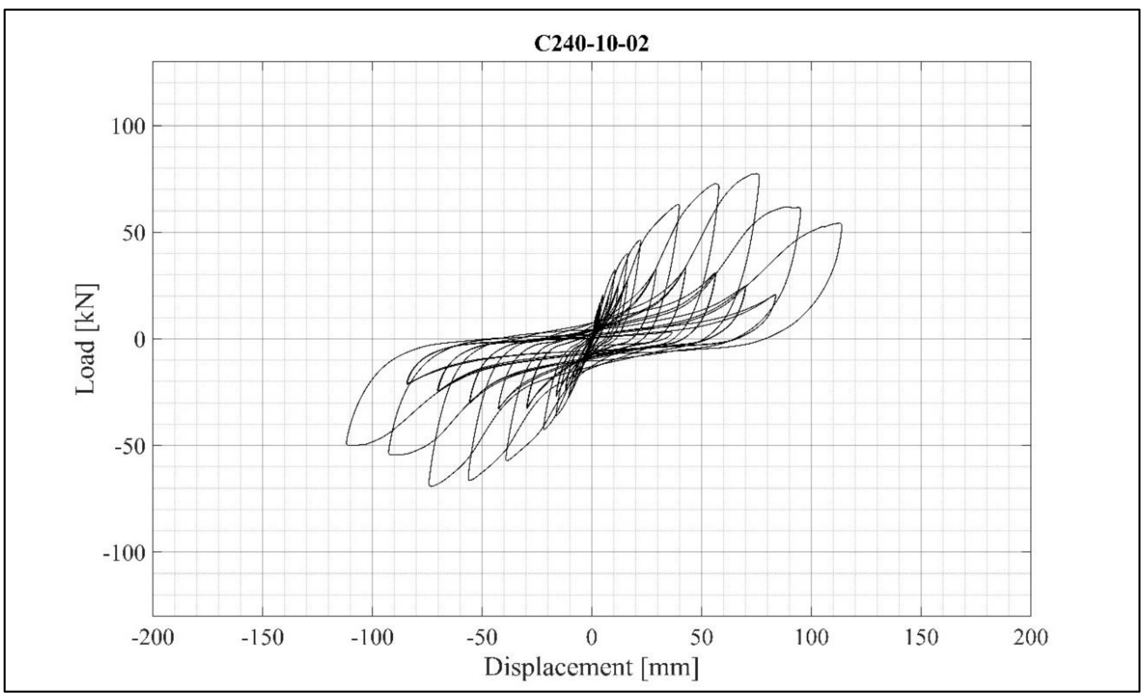

Figure D. 6. Load displacement curve of wall C240-10-02.

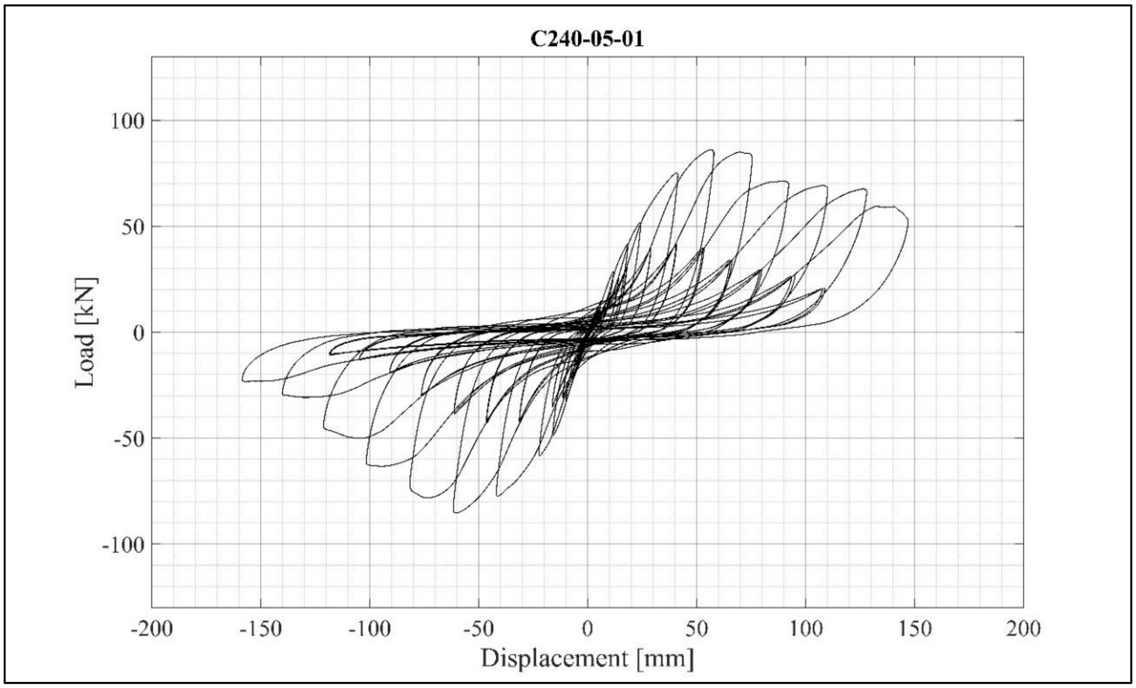

Figure D. 7. Load displacement curve of wall C240-05-01.

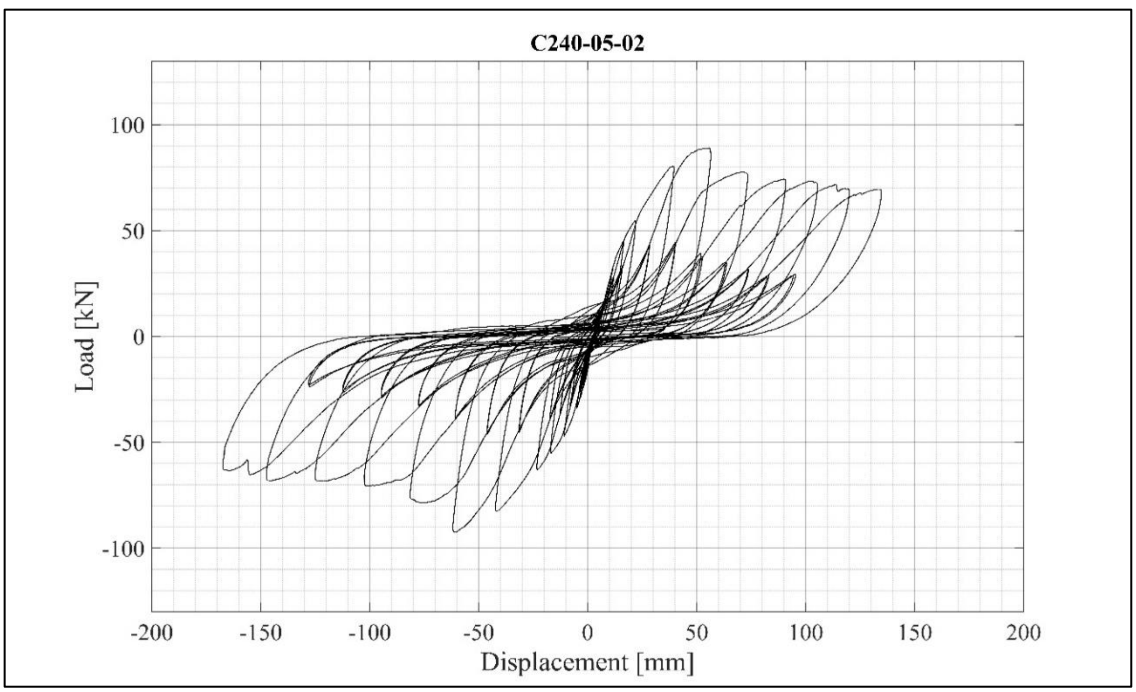

Figure D. 8. Load displacement curve of wall C240-05-02.

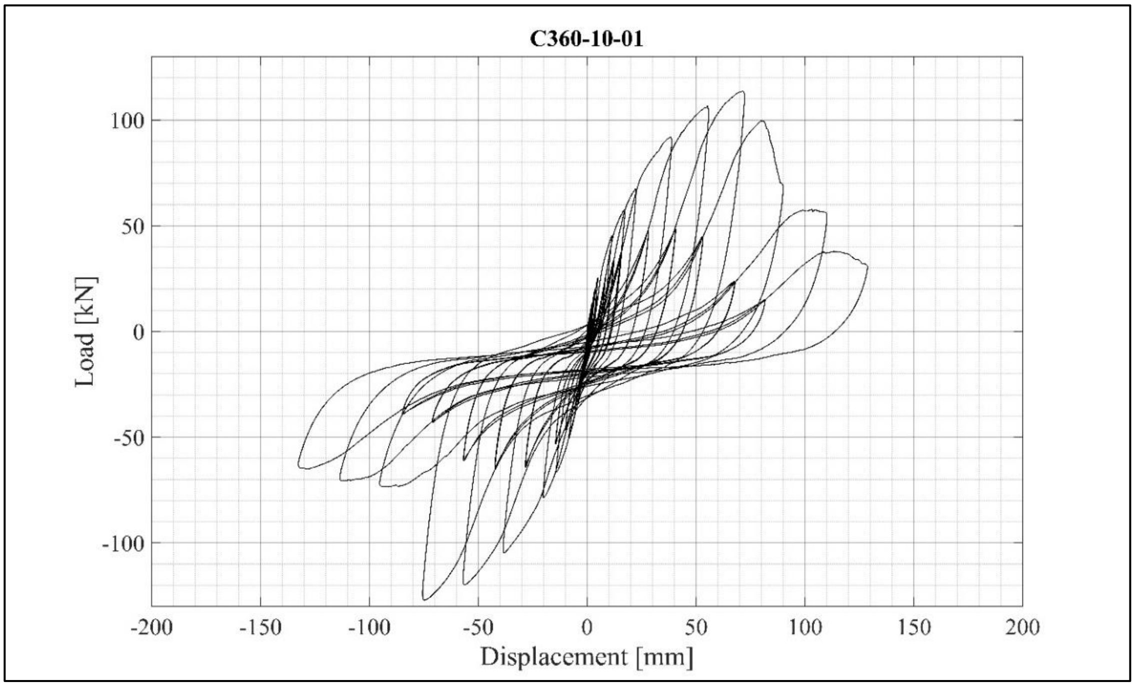

Figure D. 9. Load displacement curve of wall C360-10-01.

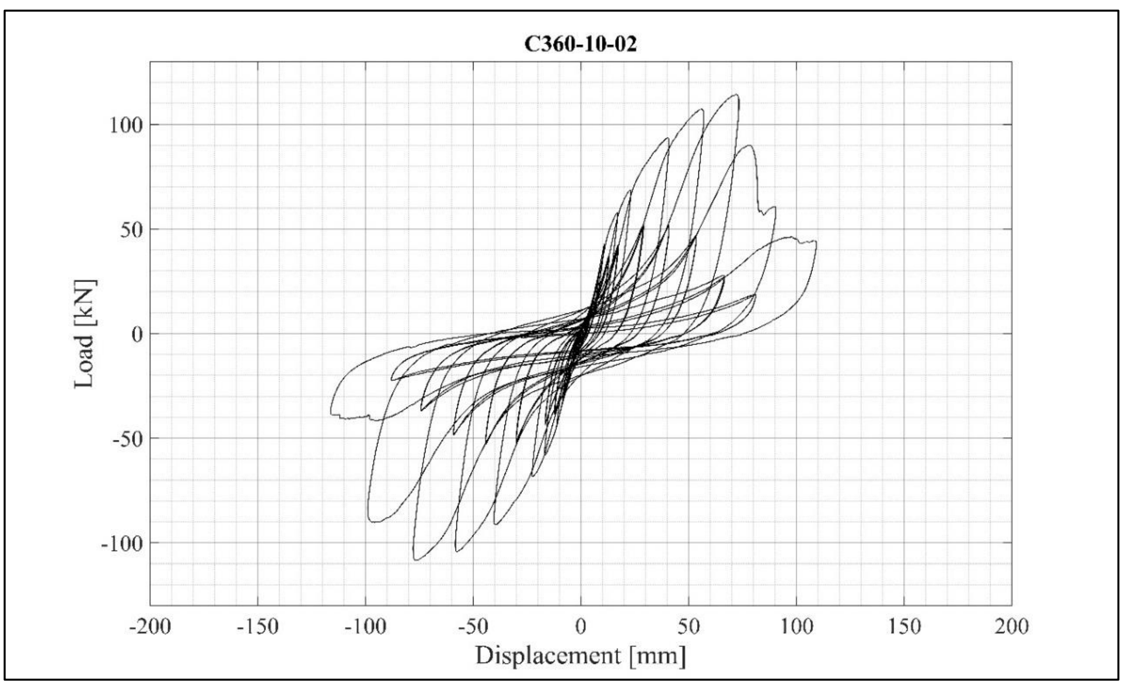

Figure D. 10. Load displacement curve of wall C360-10-02.

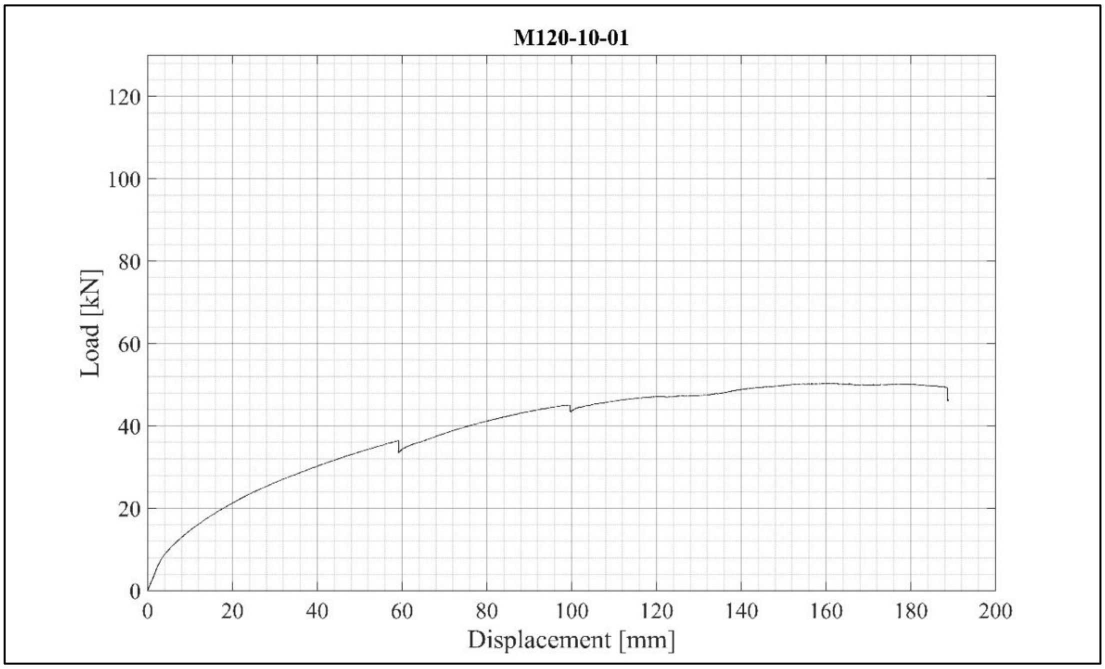

Figure D. 11. Load displacement curve of wall M120-10-01.

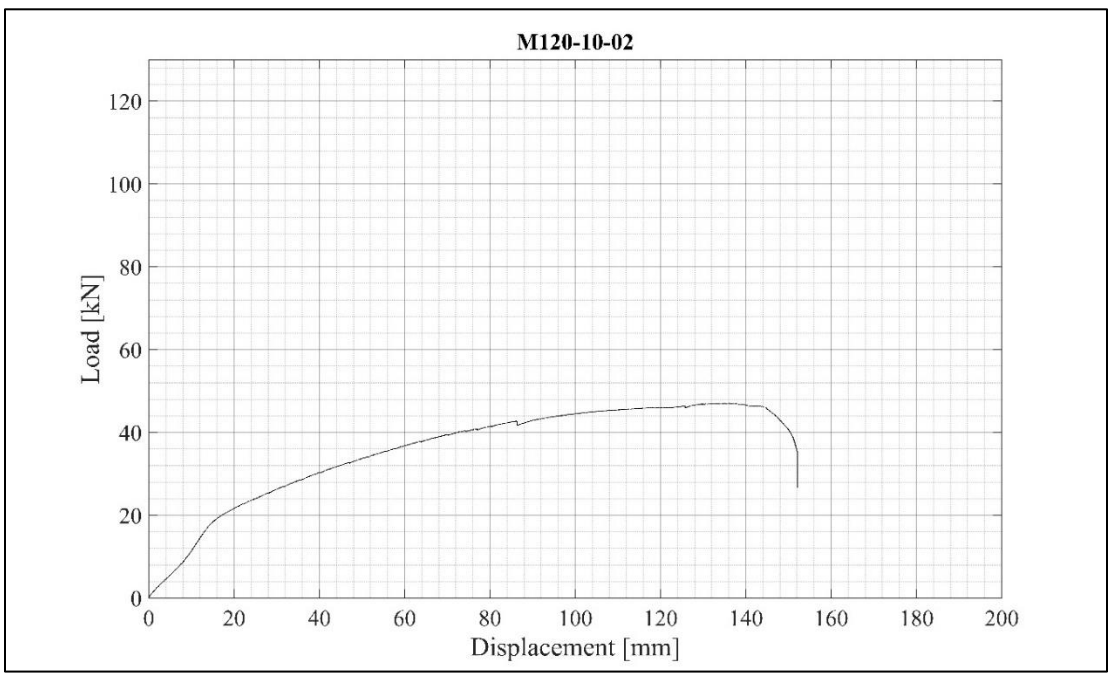

Figure D. 12. Load displacement curve of wall M120-10-02.

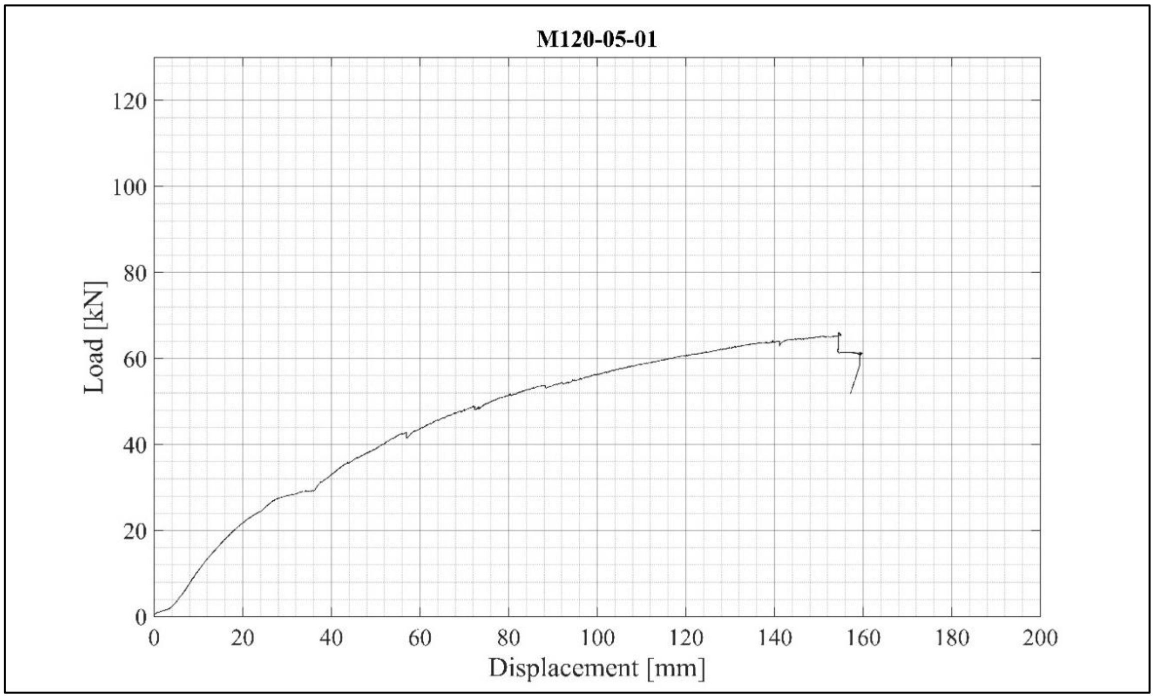

Figure D. 13. Load displacement curve of wall M120-05-01.

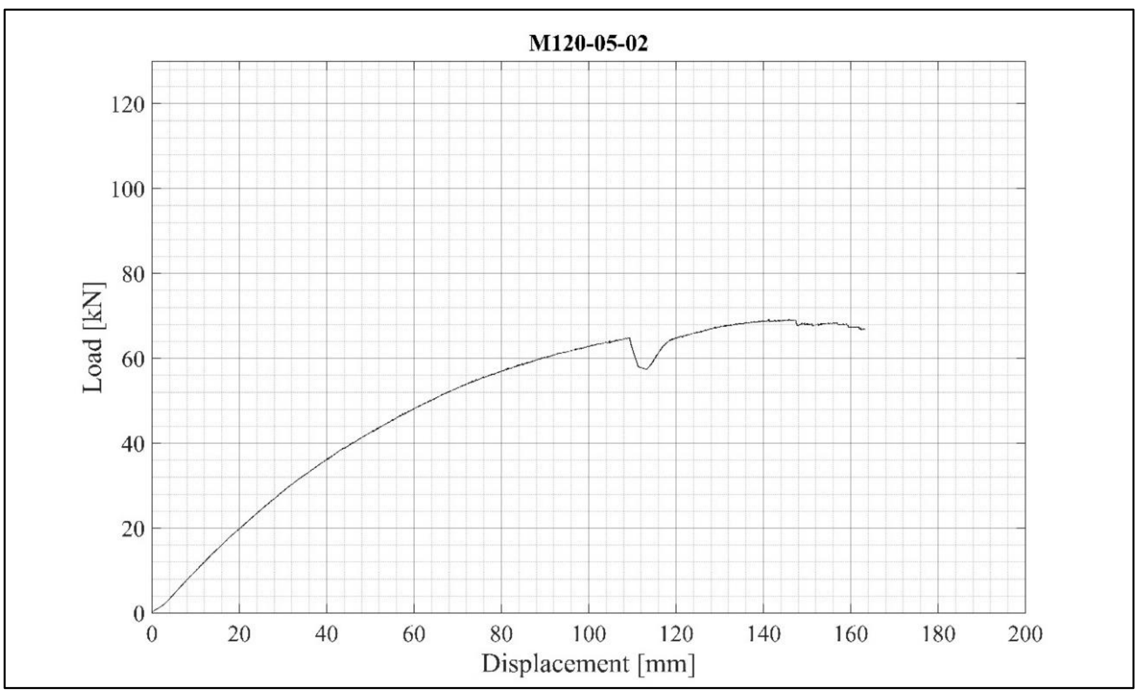

Figure D. 14. Load displacement curve of wall M120-05-02.

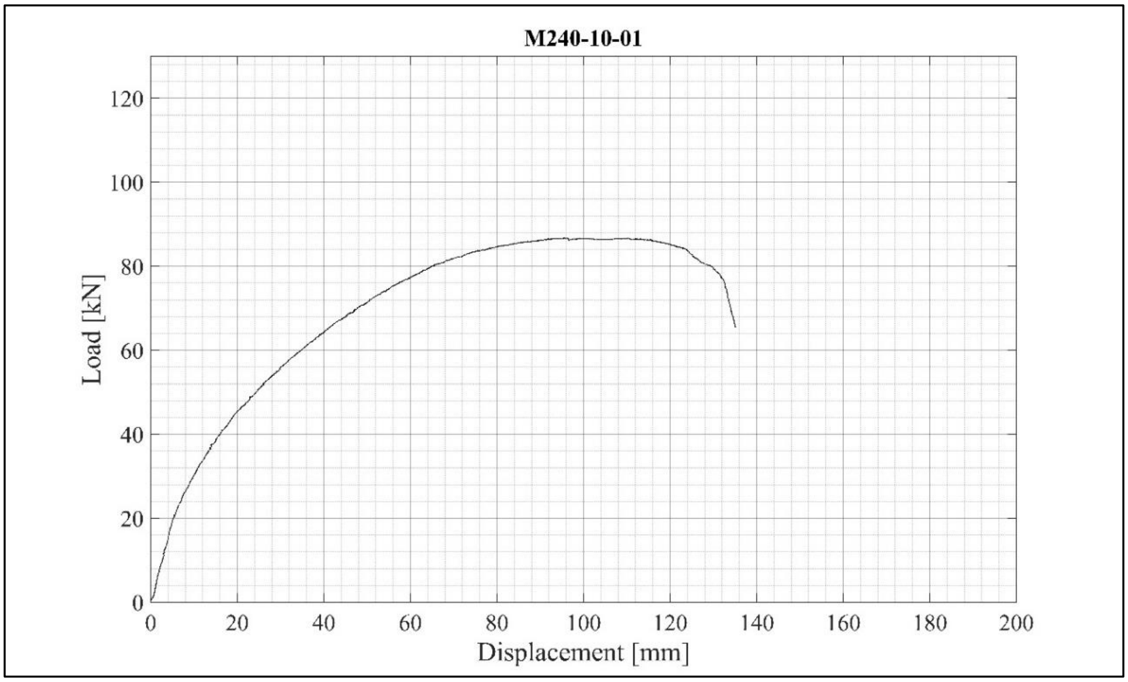

Figure D. 15. Load displacement curve of wall M240-10-01.

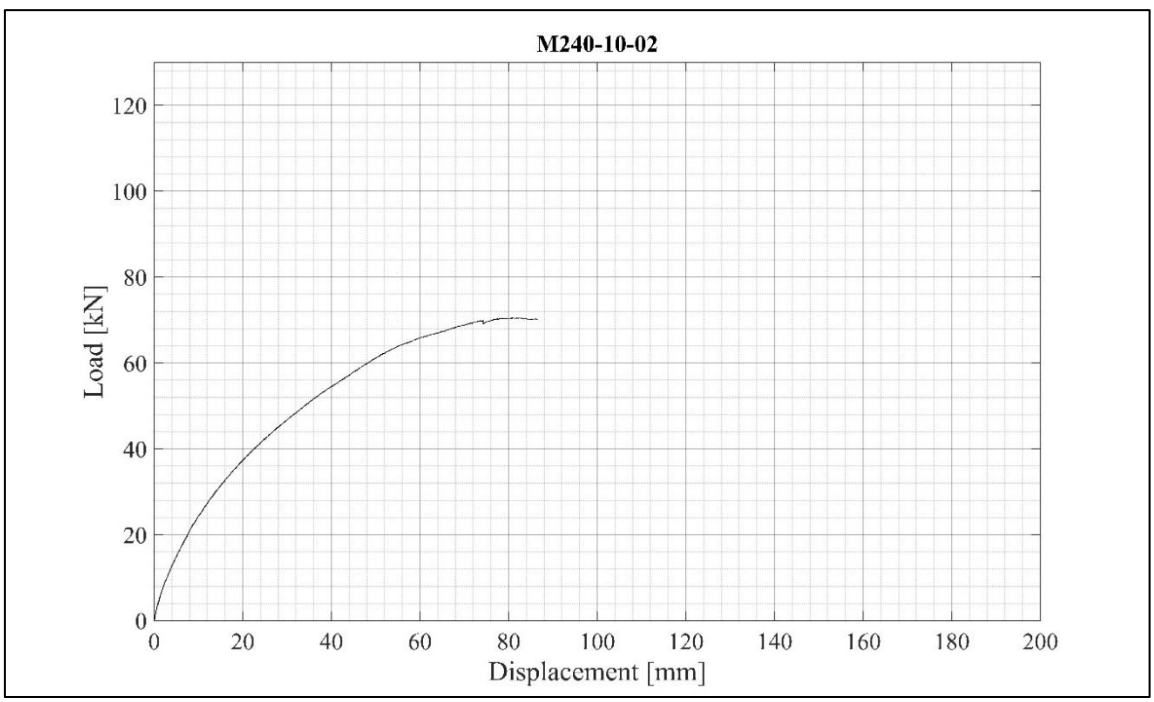

Figure D. 16. Load displacement curve of wall M240-10-02.

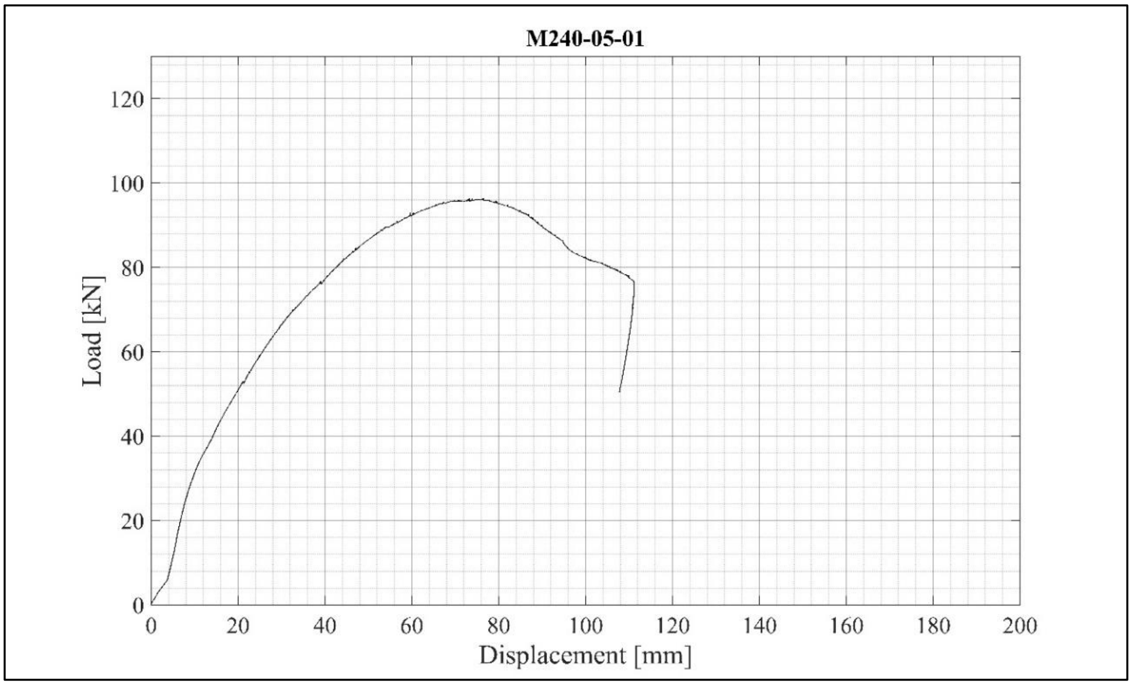

Figure D. 17. Load displacement curve of wall M240-05-01.

APPENDIX E: SHEAR DISPLACEMENT CURVES (FROM DIAGONAL TRANSDUCERS)

These values of displacements were measured according Figure 4-3.

The displacement dshear1 is obtained from measurement of transducer D09 and is positive when hydraulic actuator push, while dshear2 is obtained from measurement of transducer D08 and is positive when hydraulic actuator pulls (see instrumentation of the wall in Figure A. 1). Irregularities in the curves could be because transducers fell during the tests or some transducers were stuck.

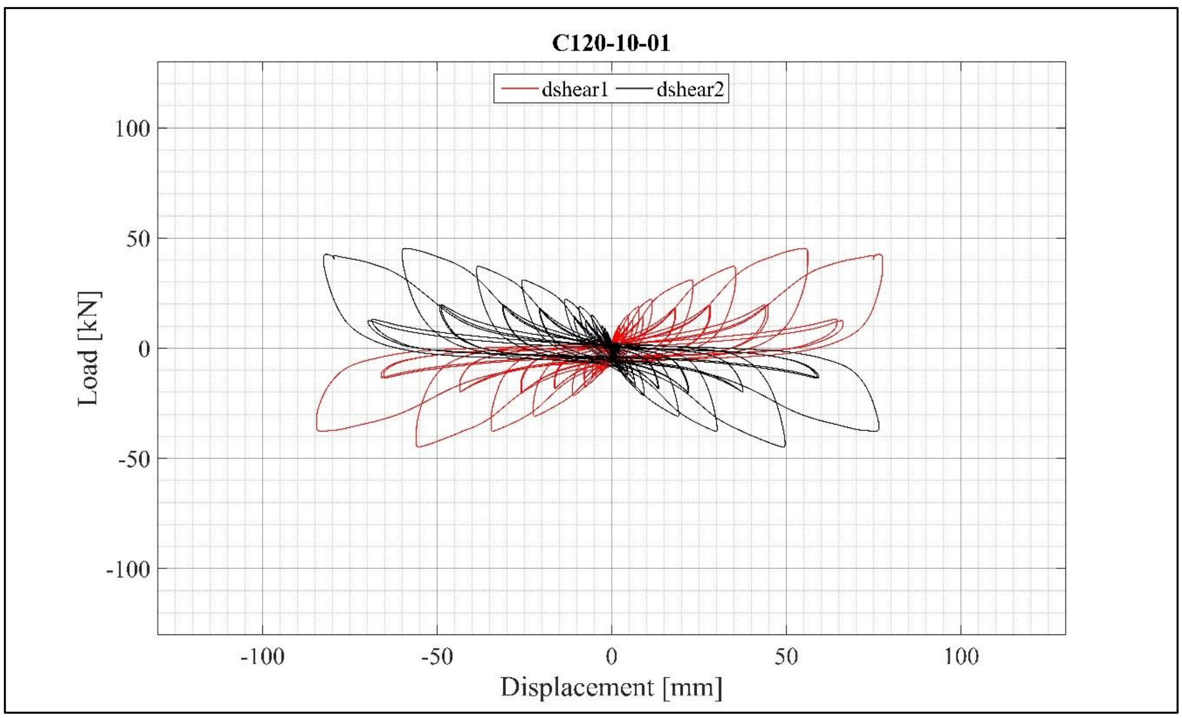

Figure E. 1. Load vs shear displacement curve of wall C120-10-01.

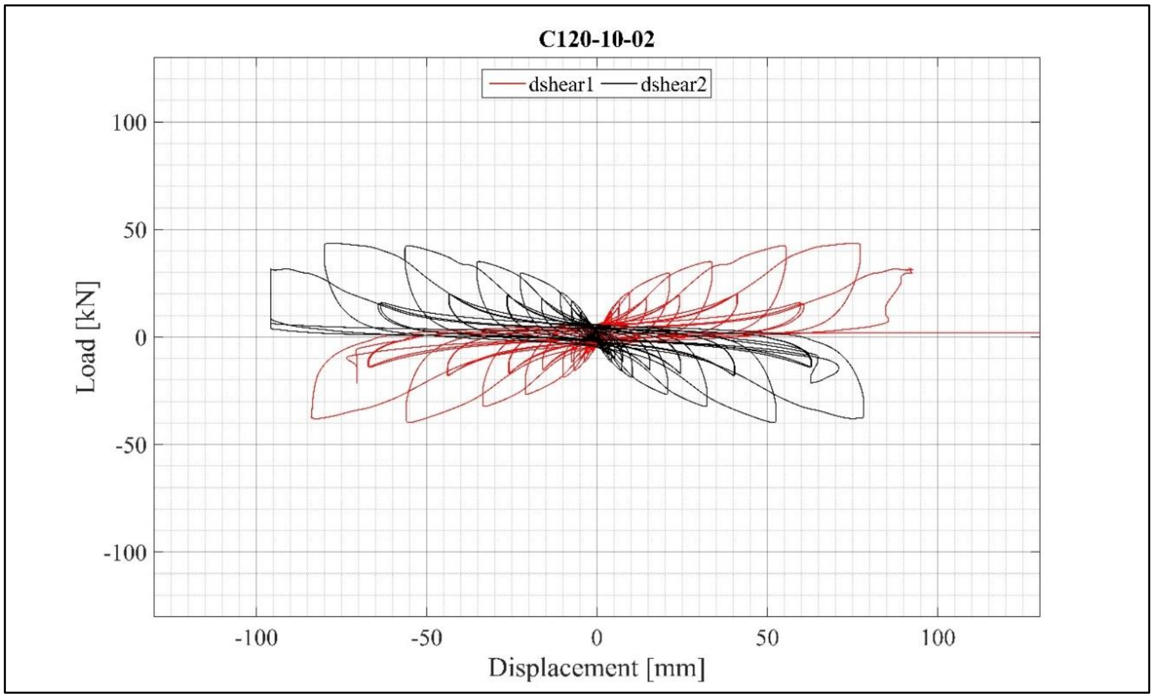

Figure E. 2. Load vs shear displacement curve of wall C120-10-02.

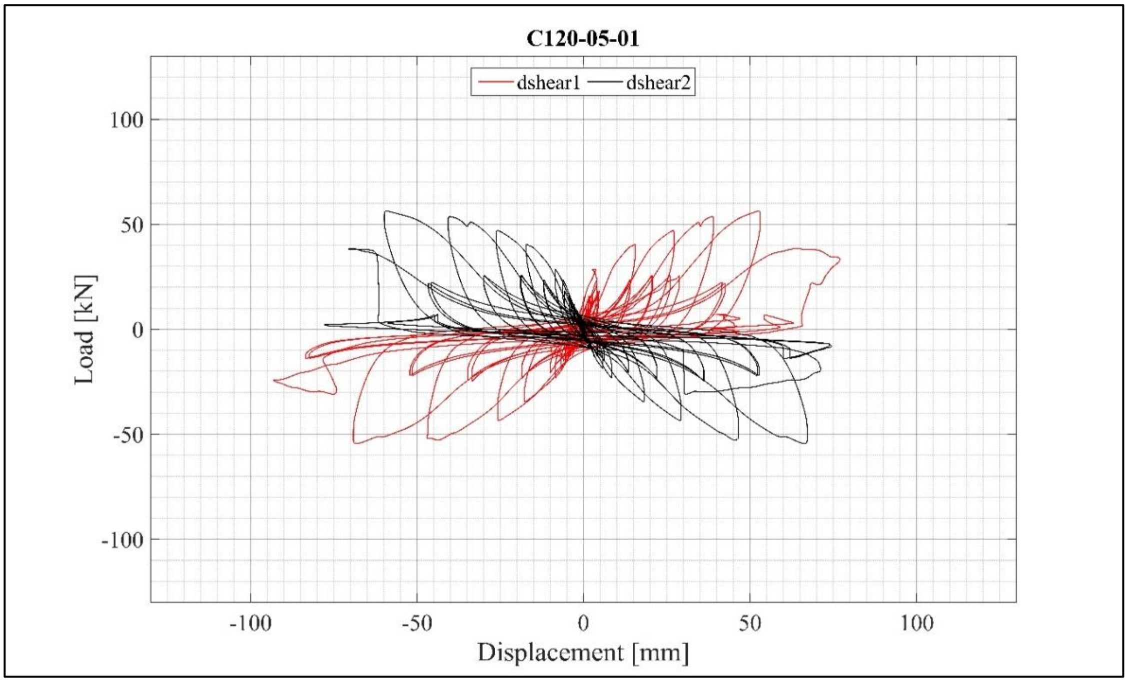

Figure E. 3. Load vs shear displacement curve of wall C120-05-01.

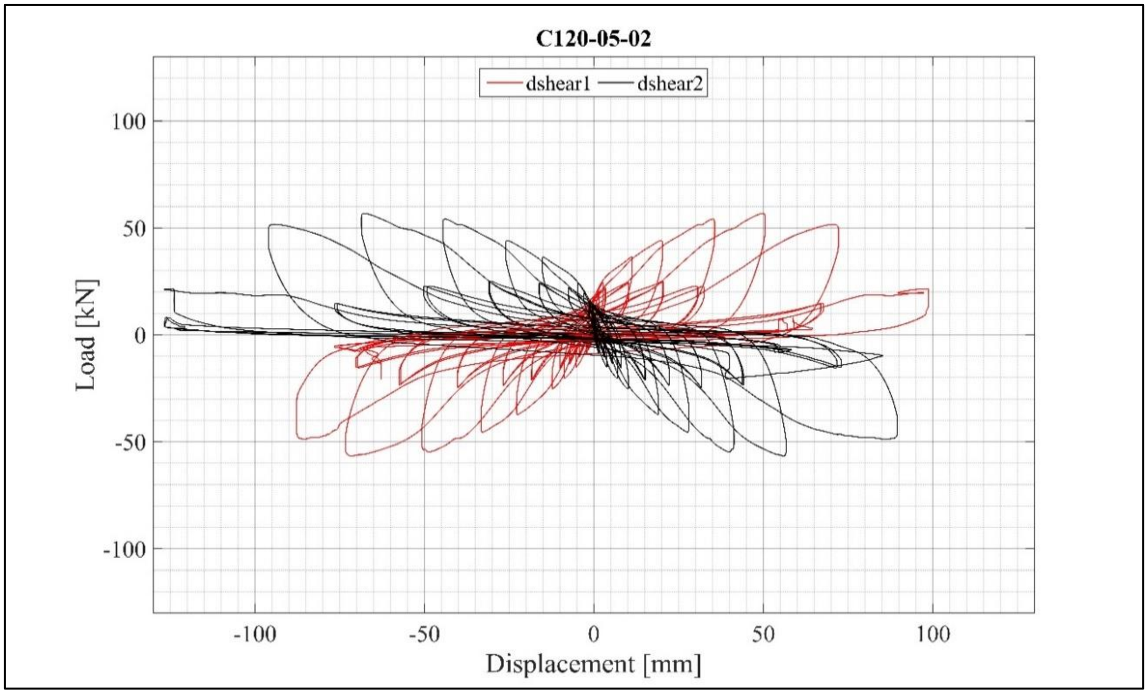

Figure E. 4. Load vs shear displacement curve of wall C120-05-02.

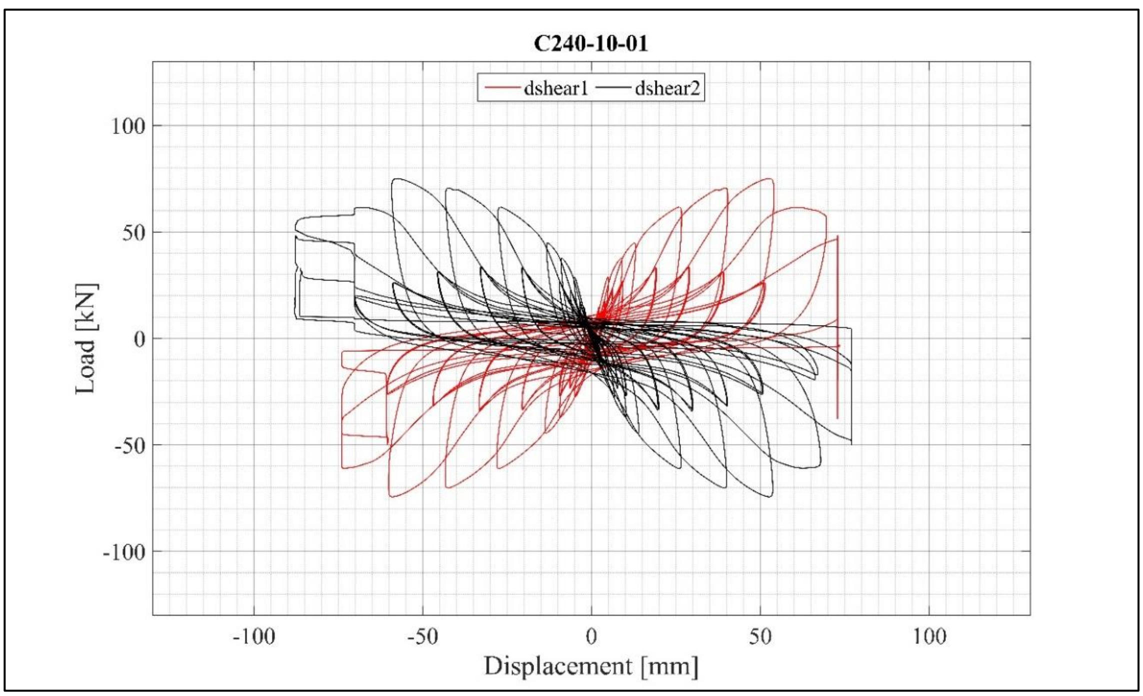

Figure E. 5. Load vs shear displacement curve of wall C240-10-01.

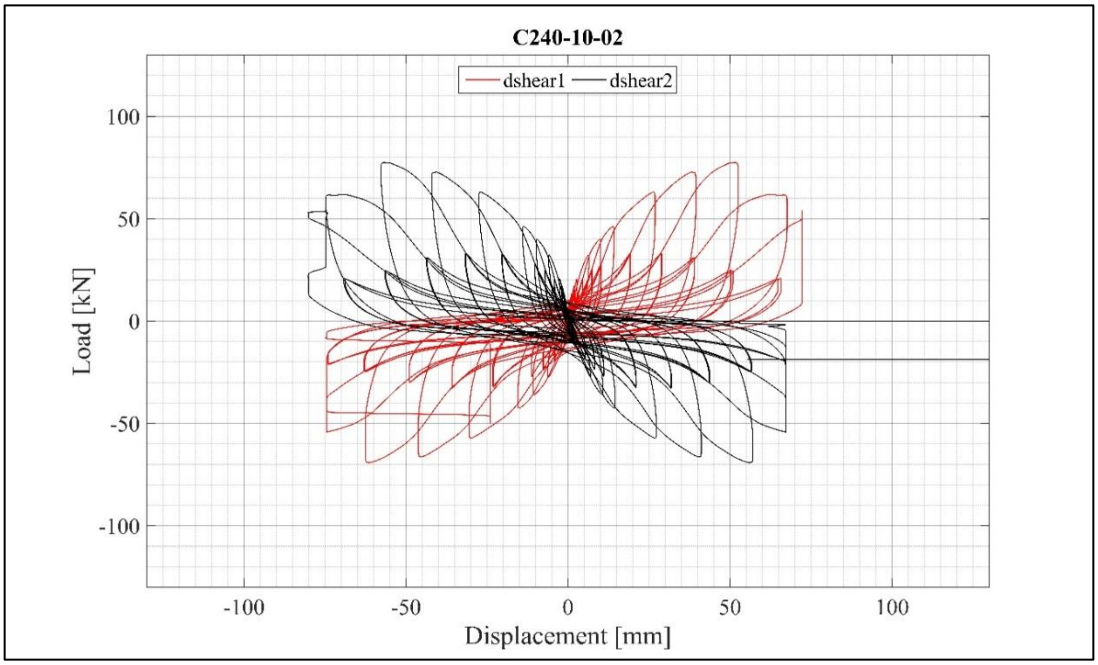

Figure E. 6. Load vs shear displacement curve of wall C240-10-02.

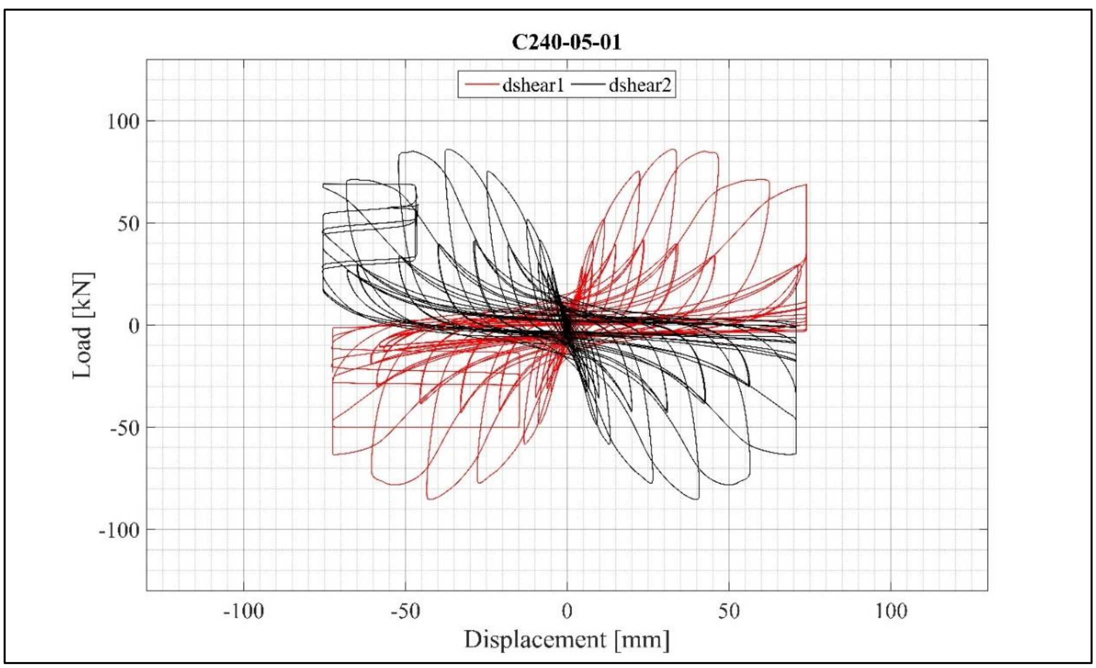

Figure E. 7. Load vs shear displacement curve of wall C240-05-01.

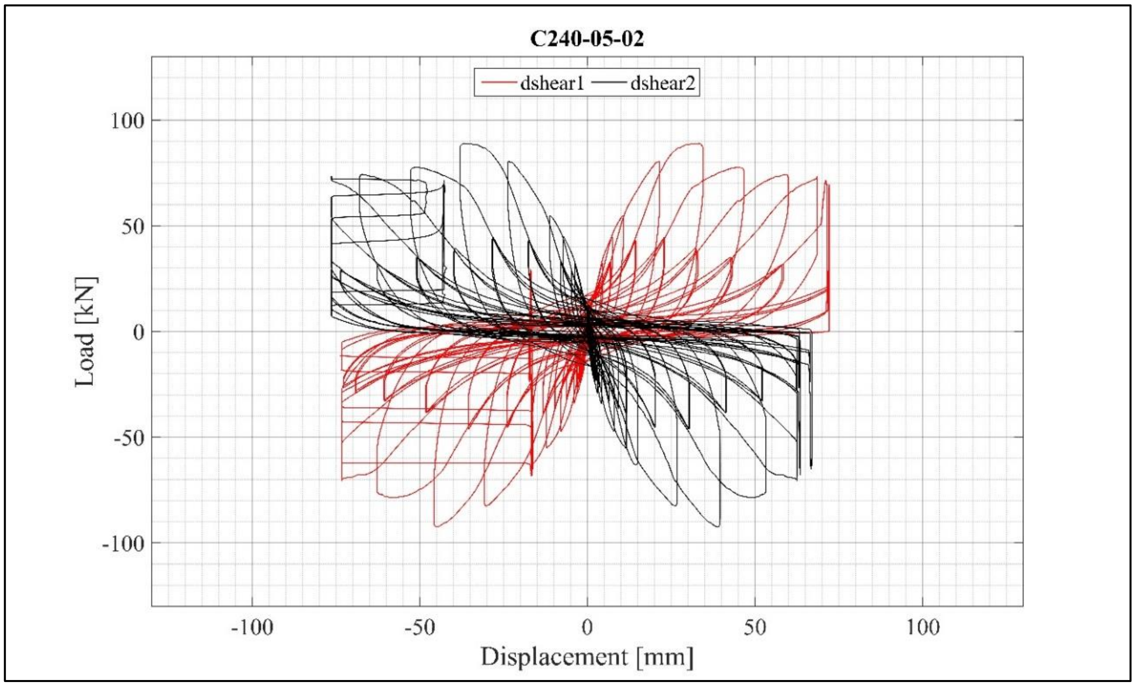

Figure E. 8. Load vs shear displacement curve of wall C240-05-02.

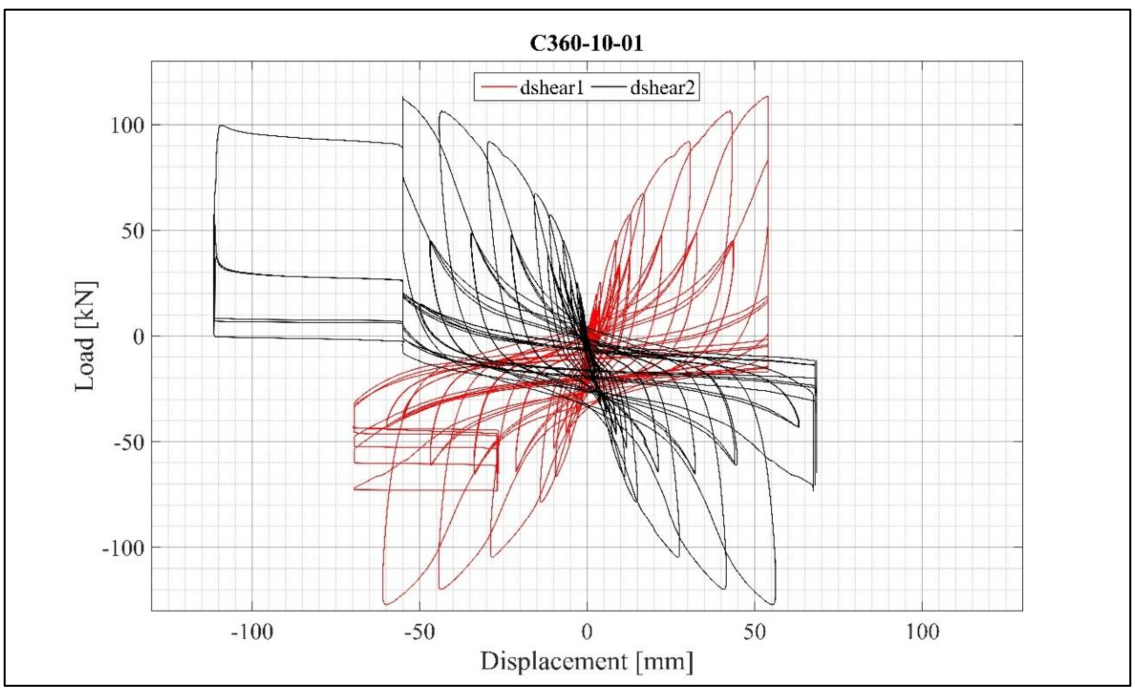

Figure E. 9. Load vs shear displacement curve of wall C360-10-01.

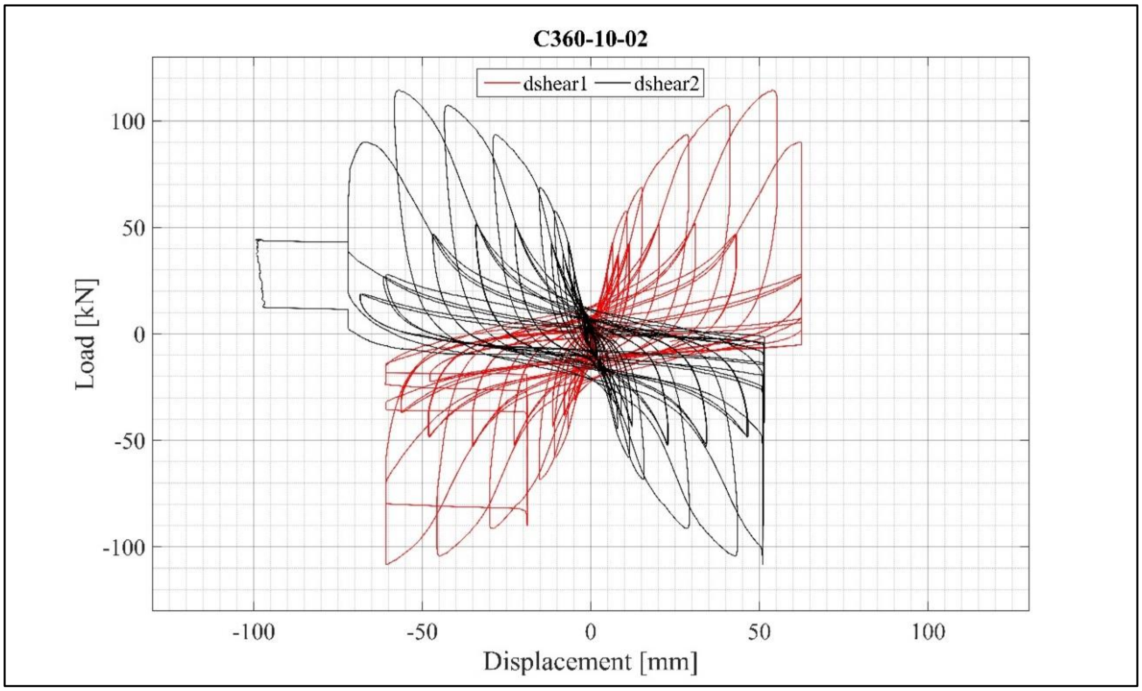

Figure E. 10. Load vs shear displacement curve of wall C360-10-02.

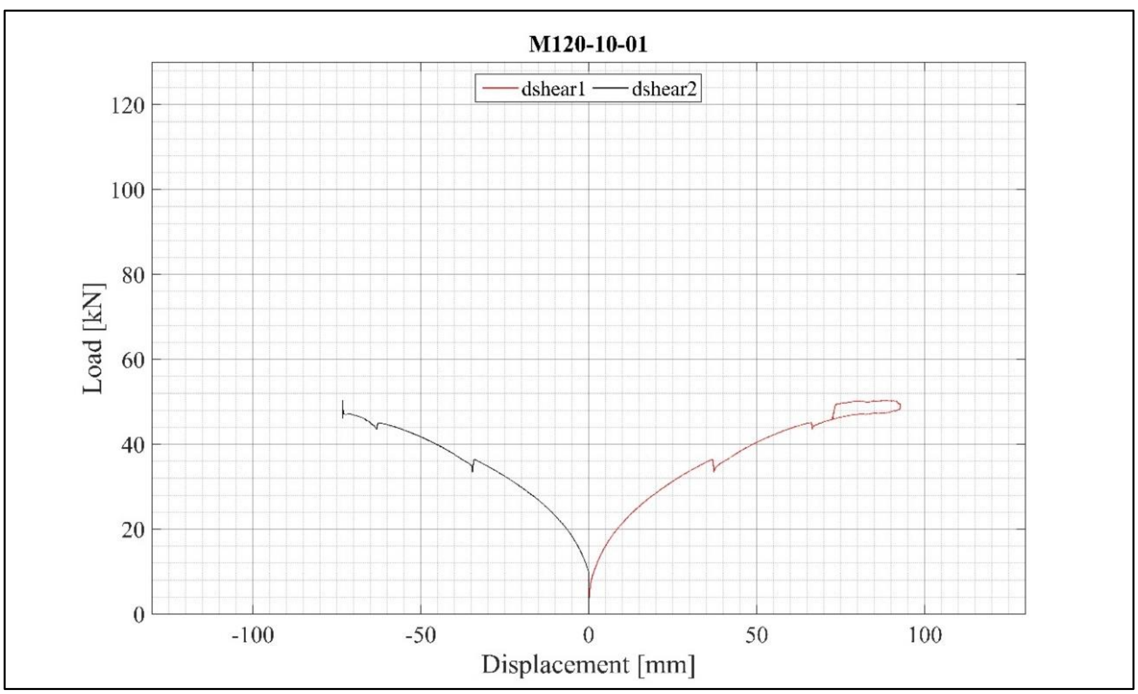

Figure E. 11. Load vs shear displacement curve of wall M120-10-01.

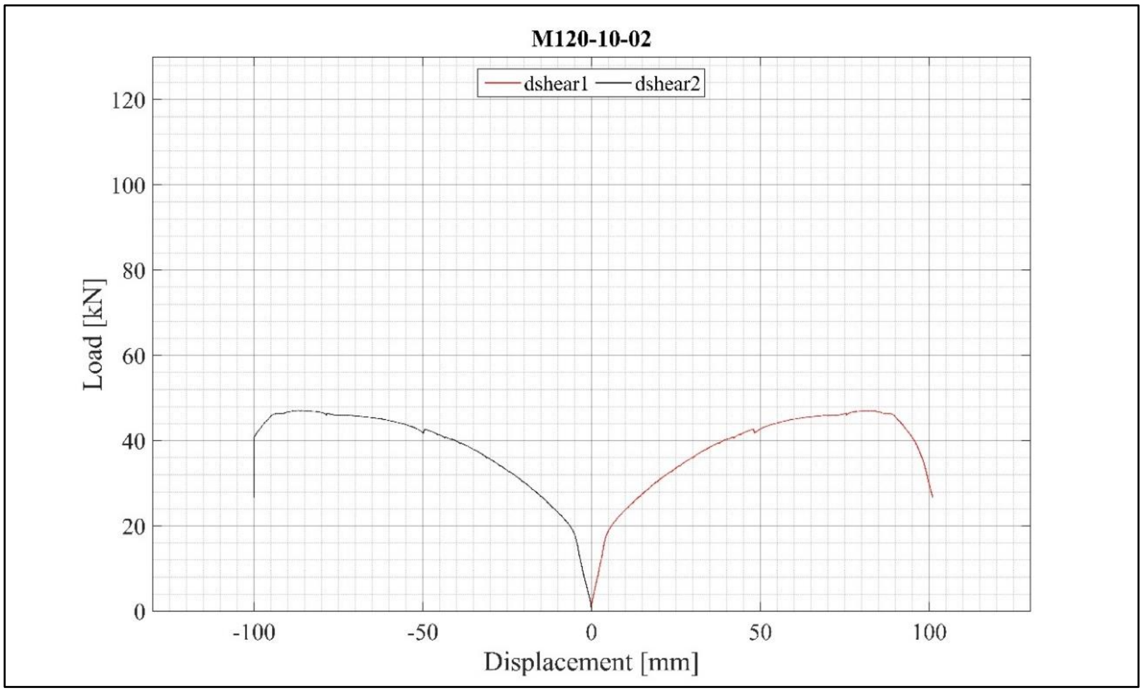

Figure E. 12. Load vs shear displacement curve of wall M120-10-02.

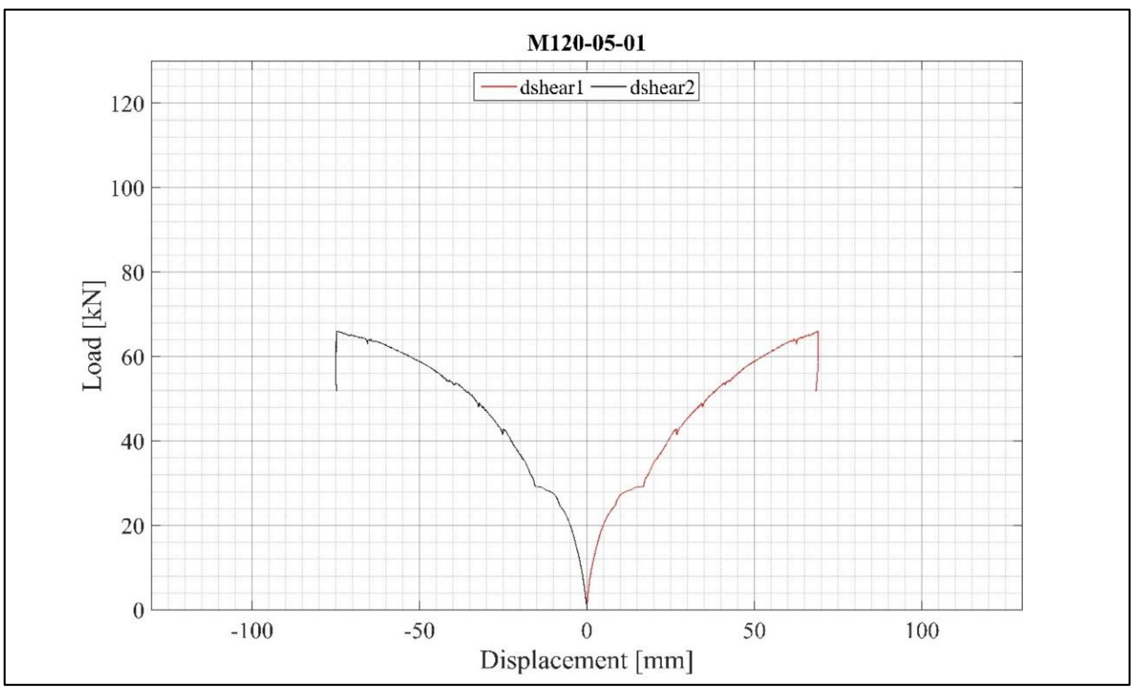

Figure E. 13. Load vs shear displacement curve of wall M120-05-01.

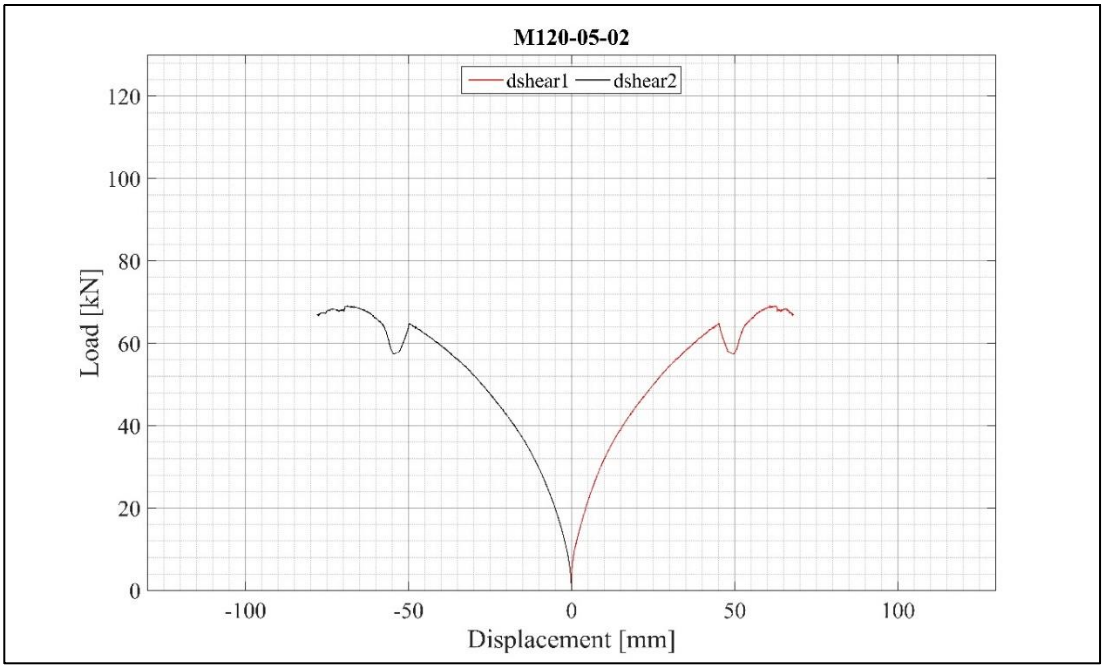

Figure E. 14. Load vs shear displacement curve of wall M120-05-02.

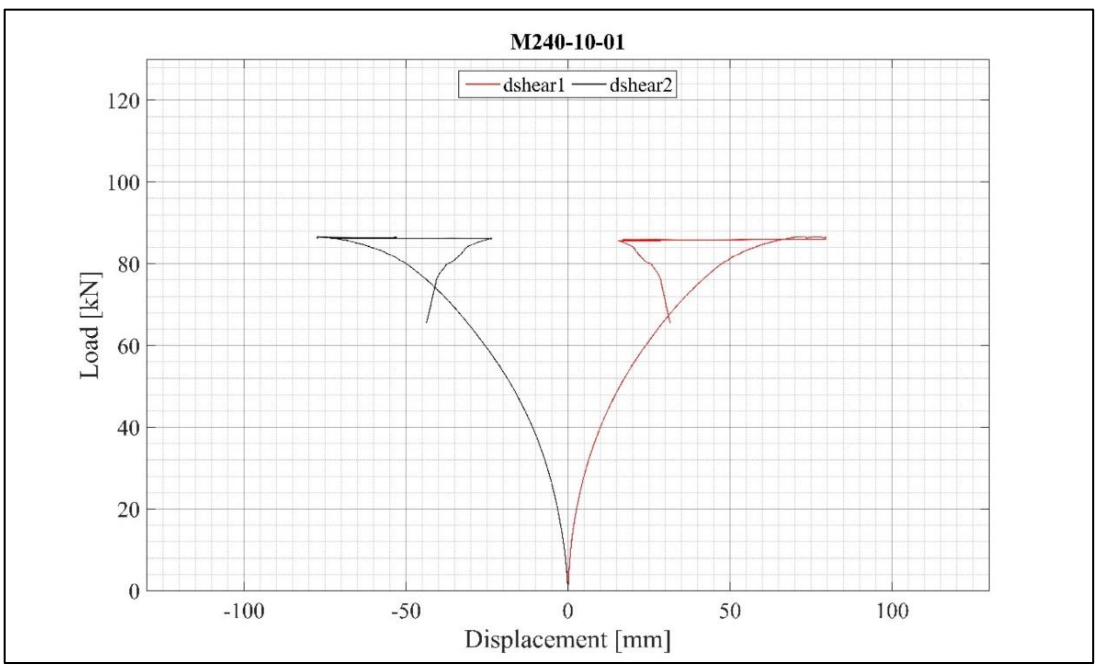

Figure E. 15. Load vs shear displacement curve of wall M240-10-01.

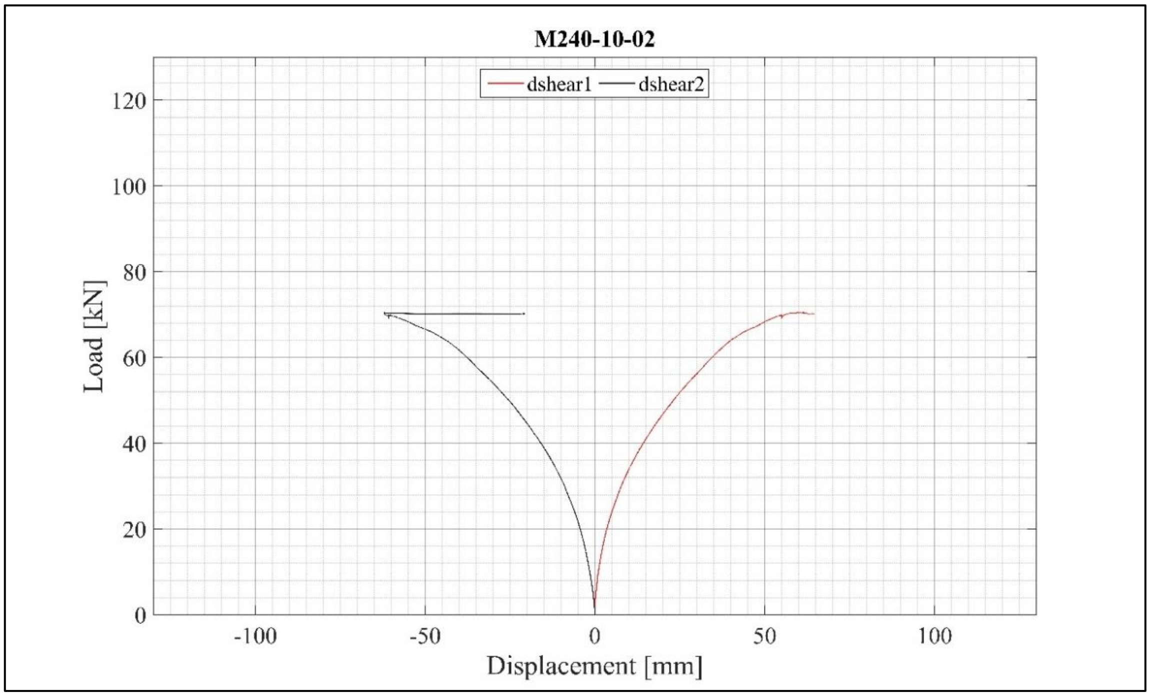

Figure E. 16. Load vs shear displacement curve of wall M240-10-02.

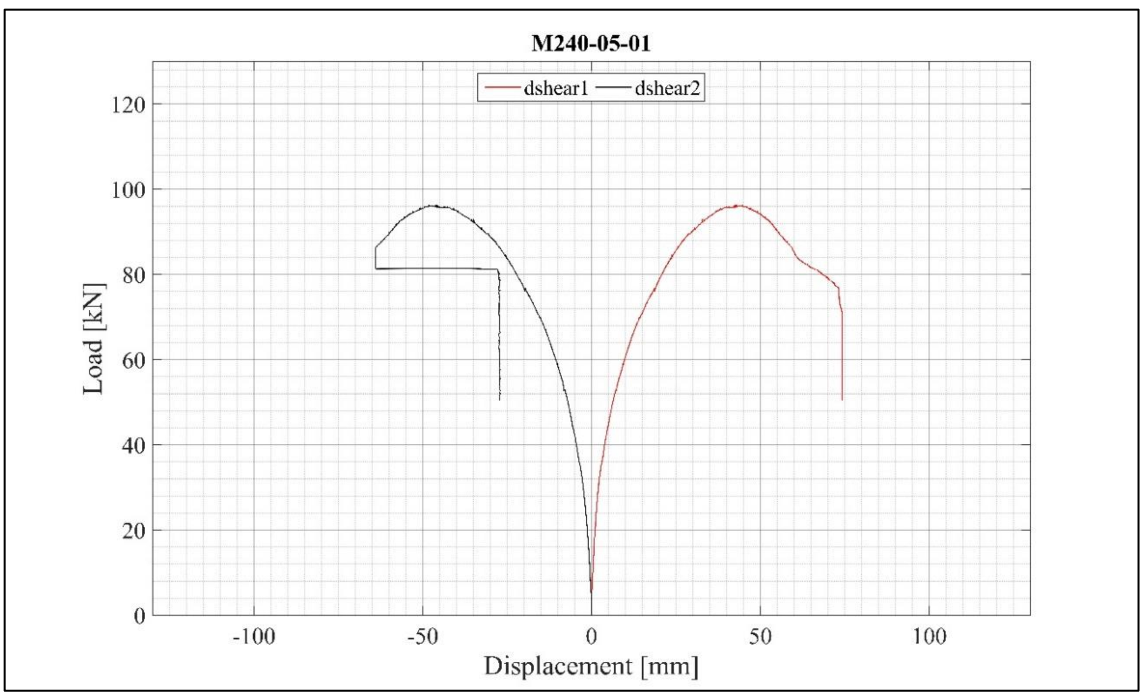

Figure E. 17. Load vs shear displacement curve of wall M240-05-01.