

Behavior under lateral cyclical load of thin reinforced concrete walls of the industrialized system

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ABSTRACT

Reinforced concrete wall buildings of the industrialized system widely used in Colombia for housing construction in recent years present particular characteristics that make them a research subject. The reduced thicknesses of their walls that give rise to high slenderness ratios, the use of nonductile reinforcement arranged in a single curtain, and the absence or low level of confinement at the ends of the walls are some of the most relevant characteristics. This article presents the results of five quasi-static cyclical tests of thin-reinforced concrete walls (TRCW) on a full-scale, which are representative of the industrialized system. Wall properties were selected from a database of 136 existing buildings. These specimens were built with different cross-sections, lengths, and reinforcement contents at the ends. The experimental results confirm the importance of the properties and amount of web reinforcement and the effect of the use of properly confined boundary elements. The observed response and analysis of the experimental database of TRCW suggest that some walls could present a limited displacement capacity compared to the maximum drifts allowed by the design code.

1. Introduction

Thin-reinforced concrete walls, named for their reduced thickness and slenderness, are the elements that provide stiffness and resistance against lateral and gravity loads in the industrialized wall system. This structural system has walls in both directions and reinforced concrete solid floor slabs with a thickness between 100 and 120 mm. These buildings generally do not exceed 15 stories in height and their wall density, calculated as the ratio between the cross-sectional area of the walls and the plan area of the first floor, varies between 1% and 7.7%, with an average of 2% and 2.5% in the longitudinal and transverse directions, respectively according to the statistical analysis of a database of 136 buildings located in a high seismic hazard zone [1–2]. The most important characteristics of these constructions are i) wall thicknesses between 80 and 150 mm, ii) low amounts of nonductile electro-welded reinforcement in the web of the walls arranged in a single layer, and iii) absence of effective confinement in the wall ends that require special boundary elements [3,4,5]. The particularities of these buildings correspond to the tendency of the builders to optimize resources and to the fact that the design codes NSR-98 [6] and NSR-10 [7] did not require a minimum thickness for the walls and boundary elements (BE).

Post-earthquake field observations in Chile (2010) and New Zealand

(2010) and experimental results of tests of slender walls (with a greater thickness than the TRCWs) evidenced some conditions for which these walls could present a low performance [8,9,10,11,12,13,14]. This, in addition to the massive use of the industrialized system for housing construction, even in areas of high seismicity, motivated some numerical and experimental programs for study walls with these specific conditions. Blandón et al. [3] tested under lateral cyclic loading four 100 mm thick walls, three with T-shaped cross section and one with a small eccentric edge at the end opposite the flange. These specimens showed limited displacement capacity, between 0.59 and 1.24%, and their failure was characterized by fracture of the reinforcement due to damage concentration at the foundation-wall interface. Also, Blandón et al. [4] presented results for two 80 mm thick rectangular walls. Similarly, they reported concentration of plasticity at the interface and a plastic hinge significantly lower than expected. The displacement capacity was greater than 2%, and the authors attribute this possibly to the type of demand imposed by the displacement protocol. In Switzerland, Almeida et al. [15] reported results for two 80 mm thick walls with similar characteristics to the TRCWs and T-shaped cross-section. One of these was subjected to bidirectional loading. In both cases, considerable out-of-plane displacements were experienced for drifts close to 0.75%. This phenomenon triggered an early failure at the end of the web. Houl

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Table 1

Summary of the experimental database for thin walls.

Wall (Author)	t_w (mm)	l_w (mm)	h_w (mm)	h_w/l_w	Cross sect. ^a	M/Vl_w	ALR ^b (%)	ρ_w (%) type ^c	Longitudinal reinf. bars/ Cross sect. ^d	ρ_b ^e (%)
W4 [3]	100	2500	2400	0.96	T	2.08	5	0.27	D 0	–
W5	100	2500	2400	0.96	T	2.08	5	0.26	M 0	–
W6	100	2500	2400	0.96	T	2.08	5	0.27	D 2#4	100x100
W7	100	2500	2400	0.96	T*	2.08	5	0.27	D 2#4	100x200
W1 [4]	80	1200	2400	2	R	3.75	5	0.20	M 1#5	80x80
W2	80	1200	2400	2	R	3.75	8	0.20	M 1#5	80x80
TW1 [15]	80	2700	2000	0.74	T	3.7	4.3	0.15	D 3Ø16	80x285
TW4	80	2700	2000	0.74	T	3.7	3.3	0.15	D 3Ø16	80x285
TUE [16]	100	1050 ^f	4250	4.25	U	4.05 ^f	0	0.30	D 3Ø16	100x200
		1300				3.58				
TUF	100	1050	4250	4.25	U	4.05	0	0.30	D 3Ø16	100x200
		1300				3.58				
MQE1-01 [17]	100	2650	2400	0.91	R	0.93	4	0.18	M 3#4	100x300
MQE1-02	100	2650	2400	0.91	R	0.93	4	0.18	M 3#4	100x300
MQE1-03	100	2650	2400	0.91	R	0.93	4	0.18	M 3#4	100x300
MQE2-01	100	2650	2400	0.91	R	0.93	4	0.26	M 3#4	100x300
MQE2-02	100	2650	2400	0.91	R	0.93	4	0.26	M 3#4	100x300
MQE2-03	100	2650	2400	0.91	R	0.93	4	0.26	M 3#4	100x300
MF	100	2650	2400	0.91	R	0.93	4	0.28	D 3#4	100x300
M1R [5]	100	1200	2400	2	R	2.1	9	0.25	M 2#4	100x200

(a) Wall cross-section, R: rectangular; T: T-shaped; T*: pseudo I-shaped; y U: U-shaped.

(b) Axial load ratio, $N/A_g f_c$.

(c) Longitudinal reinforcement ratio and type of steel of web. D: ductile reinforcement, and M: electro-welded mesh.

(d) Longitudinal reinforcement bars at the end of the wall and cross-section considered for the calculation of reinforcement ratio.

(e) Longitudinal reinforcement ratio of the wall end.

(f) Parallel to the flanges.

et al. [16] tested two U-section walls, designed according to Colombian construction practices but with a reinforcement of higher ductility. The purpose of this study was to evaluate the possibility of out-of-plane instability, limiting deformations to avoid crushing the flange boundary ends. In Peru, Quiroz et al. [17] performed tests on seven 100 mm thick rectangular walls. The response was controlled by flexure, and the walls showed an initial crack along the interface and a moderate distribution of cracking in height. Some of these specimens exhibited failure for less than 1% drift. Numerical evaluations of the behavior buildings [18,19,20,21,22], whose actual response to an earthquake is unknown, suggest that certain aspects of the design and its structural configuration could lead to limited performance, especially in some of the taller buildings subjected to considerable seismic demands. These studies, and the obvious differences with conventional reinforced concrete walls, show that the seismic response of these buildings could differ considerably from the behavior of the traditional wall system.

In contrast to the extensive experimental evidence on slender concrete walls, with a double layer of ductile reinforcement and confinement at their ends, the database of tests with typical TRCW conditions is scarce. Table 1 summarizes the properties of thin walls with a single layer of reinforcement tested in the previously mentioned programs. Although some of these tests cover certain configurations found in this type of construction, in most cases, the walls had ductile reinforcement in the web, and the amounts of reinforcement at the ends were moderate or low. Therefore, this study provides additional experimental information on walls with higher amounts of longitudinal reinforcement at the ends, electro-welded mesh non-ductile in the web, and aspect ratios (h_w/l_w) that are different from those in the experimental database. Additionally, the results for a thin wall with properly confined special boundary elements are presented. Also, information on the M1R wall tested in a previous stage of this investigation [5] is included in Table 1 and the results section for reference purposes. This wall is also representative of the structural system and has similar characteristics to those presented in this study; however, it was tested under another displacement protocol and with a higher axial load ratio; therefore, the review of its response is relevant and complements the results presented.

This article analyzes and discusses the hysteretic behavior of TRCW with different configurations and amounts of longitudinal reinforcement

at its ends. The response of the test units was evaluated in terms of displacement capacity, damage progress, failure mode, deformation components, out-of-plane behavior, and energy dissipation capacity. The tests carried out are part of the continuation of an experimental program on the performance of these structural elements, aimed at proposing an alternative reinforcement with steel plates for walls of existing buildings that may be seismically vulnerable.

2. Experimental research

In this investigation, five reinforced concrete walls were tested, at full scale, under quasi-static cyclical loading and constant axial loading. The main characteristics of the test walls are representative of the construction of the industrialized system. One of the walls was built with confined BEs in accordance with the requirements of the Colombian Code NSR-10 [7] and ACI-318-08 [23]. The tests were carried out at the School of Civil and Geomatics Engineering of the Universidad del Valle in Santiago de Cali, Colombia.

2.1. Test walls

The geometry, the amount of longitudinal reinforcement in the ends, and the type of web reinforcement were the variables selected to evaluate the response of the TRCWs. The thickness, height, materials, cross sections, amounts, configuration and type of reinforcement of the test units were defined from the typical parameters of the database of 136 existing TRCW buildings [2]. The thickness (t_w) of the walls of 100 mm is the most common in medium-rise buildings, and the height (h_w) of 2400 mm corresponds to the typical height of a floor. The length of the walls (l_w) was defined according to the capacity of the laboratory. Two values were established for the aspect ratio (h_w/l_w), that is, 1.6 and 1.4. Non-ductile electro-welded reinforcement arranged in a single layer was used for the web of the walls, with an amount (ρ_w) of 0.25% that corresponds to the typical value and the minimum amount required by the NSR-10. The longitudinal and transverse reinforcement of the web was evenly distributed. For the longitudinal reinforcement of the ends of the walls, ductile steel was used, and the amount (ρ_b) was defined according to the values in the database. The nominal compressive strength of the

concrete (f'_c) obeyed the most common value specified in the designs. The construction process was carried out following the common practices for this structural system, considering the use of dowel bars, lap splices, type of concrete, formwork, casting and curing procedures.

In this experimental program, a wall of rectangular cross-section (M3R), and three wall T-sections (M2T, M5T, and M6T), with flange dimensions of 350 mm long (l_f) and 100 mm thick (t_f) were tested. In addition to another wall with an I-section, with BEs of 250×250 mm at each end (M4I). The amount of transverse reinforcement for the confinement of the BEs of this wall was equivalent at both ends and was supplied in accordance with the requirements of Equations C.21-7 and C.21-8 of Code NSR-10, employing hoops and crossties. Fig. 1 shows the cross-section of the test units. The numbering of the walls began in M2T since the M0R and M1R walls were tested in the first stage of this investigation [5]. Although it is not common to find thin walls with confined ends and a thickness greater than the web in the industrialized system, the M4I wall was included to evaluate the effect of confinement. All the units were reinforced with electro-welded mesh in the web, except for the M2T, in which ductile steel bars were used to verify the effect of this type of reinforcement on the progress of the damage and on the instability out-of-plane. The dimensions and the reinforcement of the flanges, in the case of T-walls, do not strictly correspond to the real conditions of a wall in a building. However, the reinforcement used and the length of the flange, selected according to the constraints of the test setup, allow us to evaluate the effect of the coupling of a wall with a perpendicular element [15]. In Table 2, the properties of the test walls are summarized.

The configuration of the characteristics of the test walls allowed evaluation of some of the conditions that most influence the behavior of the walls in this system, that is, a high slenderness ratio ($h_w/t_w > 16$) that can cause instability out of plane [15,24], a low amount of nonductile reinforcement in the web which can lead to damage concentration at the foundation-wall interface [10,22] and the absence of effective

confinement at the ends of the walls [25].

The test walls were designed for a nominal compressive strength of 21 MPa. The actual compressive strength, obtained by testing cylinders at the age of 28 days, according to ASTM C39, is shown in Table 2. Tension tests were performed for bars # 2 (6.35 mm), # 4 (12.7 mm), # 5 (15.88 mm), and # 6 (19.05 mm) and for the electro-welded mesh with a diameter of 7 mm. In Table 3, the properties of the reinforcement are summarized. The average ultimate strain, obtained as the total elongation at a gauge length of 100 mm at maximum force, was approximately 11% for bars # 4 and larger bars and 9.5% for bars # 2 used in the web of the M2T wall. The bars showed a well-defined yield zone. The electro-welded mesh, due to its manufacturing process [22], did not present a defined yield zone, and its ultimate strain was approximately 2.5%. Fig. 2 shows the typical stress versus strain curves for bars # 2 and # 5 and the electro-welded mesh.

2.2. Test setup, instrumentation, and displacements protocol

The test setup is shown in Fig. 3. The walls were anchored in the fixed base of the reaction frame. The upper beam of the wall was connected to a structure consisting of a transfer beam (HEA 400) and a rigid metal frame. This assembly allowed to increase the height of the application of the horizontal load (up to a height of 3905 mm measured from the base) to obtain a greater shear span ratio. For the execution of the horizontal displacements protocol, the rigid metal frame was connected to a hydraulic actuator with a capacity of + 500 kN and a maximum stroke of +/− 300 mm. The axial load was applied by two actuators with a capacity of + 1000 kN and a maximum stroke of +/− 100 mm. These actuators were connected by means of metallic tensioners to each of the ends of the HEA 400 beam to transmit the load to the upper beam of the wall. Hinges were used at the bottom of the actuators and the end of the vertical tensioners so as not to restrict the movement and rotation of the walls during the test. On the side beams of the reaction frame, two

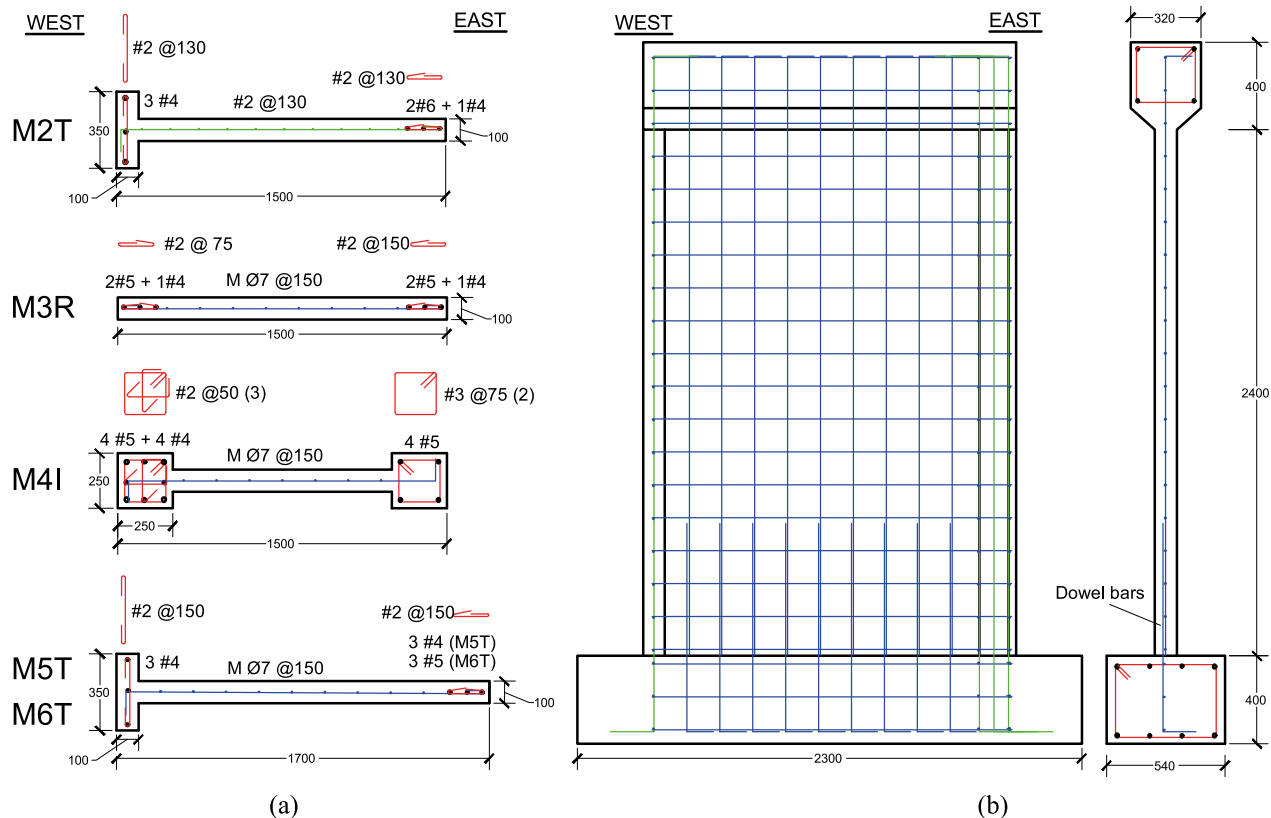


Fig. 1. (a) dimensions (mm) and reinforcement of the test walls and (b) elevation and cross section of a typical wall with flange.

Table 2
Properties of the walls test.

Wall	l_w (mm)	Web reinforcement ρ_w (%)	Longitudinal reinforcement at wall ends ρ_b (%)		f_c (MPa)	ALR (%)
			Flange or west end	East end		
M2T	1500	1 #2 @130 (0.24) ^a	3 #4 (1.09) ^c	2 #6 + 1 #4 (3.49)	24.50	4.9
M3R	1500	M 07 mm @150 (0.26) ^b	2 #5 + 1 #4 (2.62)	2 #5 + 1 #4 (2.62)	24.50	5.0
M4I	1500	M 07 mm @150 (0.26)	4 #5 + 4 #4 (2.08)	4 #5 (1.27)	24.50	5.0
M5T	1700	M 07 mm @150 (0.26)	3 #4 (1.09)	3 #4 (1.90)	25.47	5.0
M6T	1700	M 07 mm @150 (0.26)	3 #4 (1.09)	3 #5 (2.97)	25.47	4.9

(a) 1 ductile steel bar diameter (6.35 mm) spaced every 130 mm.

(b) Electro-welded mesh of diameter 7 mm spaced every 150 mm.

(c) The reinforcement ratio at the ends was estimated for the area calculated as the product of the thickness by the length measured between the outer edges of the transverse reinforcement plus the concrete cover on both sides.

Table 3

Mechanical properties of steel.

Reinforcement	f_y (MPa)	f_{max} (MPa)	E_s (GPa)	ϵ_u (%)
Mesh Ø 7 mm	596.1	662.8	193.4	2.49
Bar # 2	509.0	645.8	195.6	9.49
Bar # 4	459.9	653.1	198.4	11.1
Bar # 5	465.0	641.2	194.9	11.0

f_y : yield stress.

f_{max} : maximum stress.

E_s : modulus of elasticity.

ϵ_u : ultimate strain.

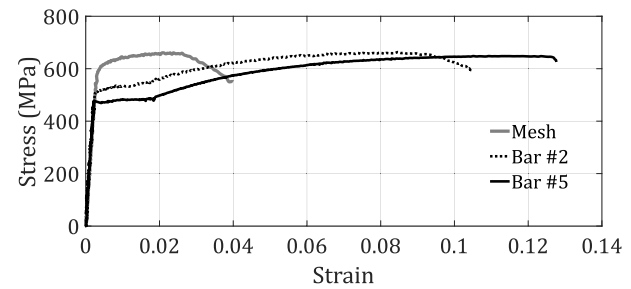


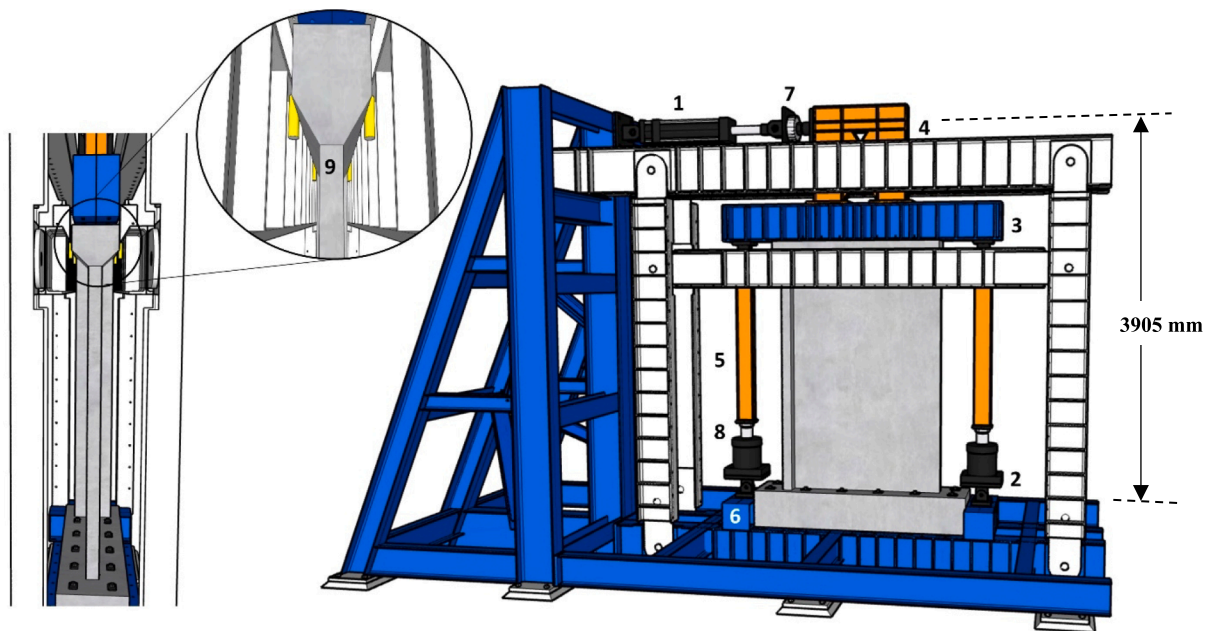
Fig. 2. Typical stress-strain curves.

lateral supports made up of metal channels and rollers were installed on each side to restrict out-of-plane displacements at the level of the upper beam of the wall.

The axial load ratio, ALR, corresponding to the relationship between the applied load and the nominal resistance to compression of the walls, $N/A_g f_c$, was kept constant in all tests, with an approximate value of 5% (Table 1). The shear span ratio ($M/V l_w$) was 2.6 for the M2T, M3R, and M4I walls and 2.3 for the M5T and M6T walls. The values of these parameters correspond to the expected demands on the buildings of this structural system in zones of high seismic hazard in Colombia. The ALR for gravity ($D + 0.25L$) for these constructions varies between 2.5% and 8%, and the shear span ratio varies between 1.3 and 3.4 [1]. (Where: D and L correspond to the dead and live loads, respectively).

The details of the typical instrumentation of a wall are shown in Fig. 4. In each test, three load cells were used to record lateral and axial loads. 35 sensors, type linear variation differential transducers (LVDTs), potentiometers, and string pots, were used to measure lateral displacements, axial deformations, shear deformations, out-of-plane displacements, and possible displacements or rotations of the foundations of the walls. During the construction of the walls, ten strain gauges were installed at the interface level and at a height of 300 mm to measure the axial deformations in the bars of the longitudinal reinforcement at the ends of the walls and in the web wires (Fig. 4b). Additionally, strain gauges were glued to the surface of the concrete on the lateral faces (east and west) and at the ends of the north face, 30 mm from the interface level and 300 mm high. The digital image correlation technique (DIC) was used to monitor the progress of the damage, for which a pattern of random points was painted on the surface of the south face of the walls, and during the tests, pairs of images (left and right) of the condition of the wall were taken every 10 to 20 s with a stereo set-up of cameras.

In the tests, the axial load was applied and kept approximately constant by controlling the hydraulic pressure in the actuators. Subsequently, the horizontal load was applied, controlled by displacements, through the implementation of the protocol shown in Fig. 5. The levels of drift ($\Delta = \delta/h_w$) of the protocol were defined by numerical simulations of the expected response of the walls [20,26] to properly capture the behavior in the elastic range and the progress of the damage until failure. For each level of drift, the walls were subjected to two cycles of displacement. The drifts defined for the protocol were 0.025% (0.6 mm), 0.05%, 0.1%, 0.2%, 0.4%, 0.6%, 0.8%, 1.0% (24 mm), 1.2%, 1.4%,



1. Lateral load actuator, 2. Vertical load actuators, 3. Transfer beam, 4. Rigid metal frame, 5. Tensioners system, 6. Side supports, 7. Lateral load cell, 8. Vertical load cells, and 9. Side rollers.

Fig. 3. Test setup.

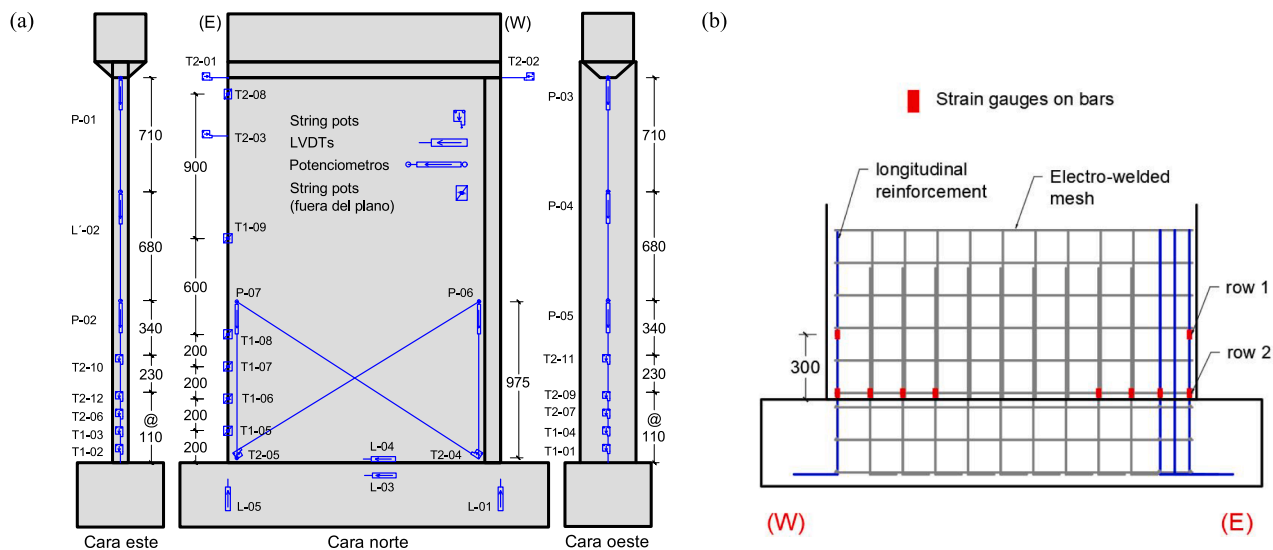


Fig. 4. (a) Instrumentation of a test wall on the north face and lateral faces, and (b) location of internal gauges on the bars.

1.6%, 1.8%, 2.0% (48 mm), 2.25%, 2.5% and so on until failure. The speed of execution of the test was controlled to avoid dynamic effects, allow the review of the operation of the hydraulic system, and record the damage to the walls. It was defined as the positive direction of the load when the east end of the walls was subjected to compression.

3. Test results

3.1. Force vs drift relationship

The lateral force versus drift hysteresis curves are presented in Fig. 6. The drift was calculated as the relationship between the lateral displacements measured at the lower level of the upper beam of the wall

and the height of the walls (2400 mm). For the evaluation and comparison of the behavior of the units, four limit states were defined: I) first cracking, estimated for the range of drifts in which the stiffness of the wall is maximum [3], II) the first yield in the longitudinal reinforcement, obtained from the records of the strains in the reinforcement bars of the ends of the walls, III) maximum resistance, and IV) ultimate resistance or failure, defined as the peak lateral load reached during the test and the instant in which the lateral force degrades approximately 20% of the maximum resistance [27]. In Fig. 6f, the backbone curves of the five walls are shown, and the M1R wall tested in the first stage of this investigation is included [5]. This wall, with a rectangular section, had similar characteristics to the M3R, but with a length of 1200 mm, an amount of longitudinal reinforcement at the ends of 1.46%, a shear span

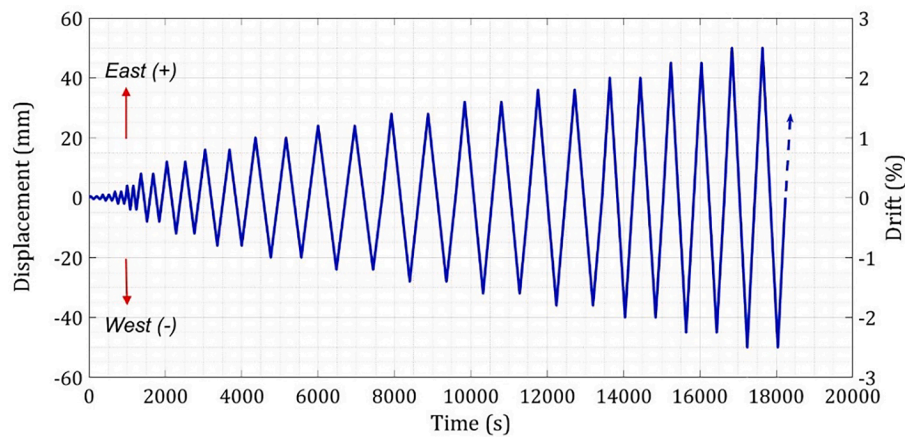


Fig. 5. Displacements protocol.

ratio of 2.1, and an ALR of 9%. In Table 4, the drift and the shear stress normalized by the square root of the compressive strength of the concrete, f_c , are listed for each limit state. The shear stress (τ) was calculated as the relationship between the lateral force measured in the test and the area of the web ($lw.tw$).

3.2. Damage progress and failure mode

The damage was recorded manually by marking the cracks and measuring their thickness on the north face of the walls and by means of the DIC technique on the south face. Table 5 shows a summary of the most significant events recorded during the tests and the level of drift on which they occurred. Limit states, defined in the previous section, are included to show the sequence of damage and compare the responses of the walls. In Fig. 7, images of the walls are shown near the state of ultimate strength in both directions, indicating the cracking pattern (not the actual crack width) and the extent of the damage. The figures show the vertical strains calculated by image processing.

The cracking pattern was similar in both directions on all walls. Except for the M4I, the walls show a similar tendency in the first two limit states. Cracking (I) was reached for early drifts, less than 0.1%, and the first yield (II) was recorded for the drift level of 0.2%. Regarding the states of maximum resistance (III) and ultimate (IV), the capacity and ultimate drift of the walls depended on the properties and quantity of reinforcement used and the configuration of their ends.

The response of the walls was dominated by a bending behavior. The walls exhibited fracture of the longitudinal reinforcement of the web, damage in the compressed region at both ends, and buckling of the reinforcement. In all the units, a considerable number of secondary cracks were formed and distributed along its height, and a main crack in the interface foundation-wall that was prolonged and widened considerably causing the detachment of the base. From this point, rigid body rotations were observed. This mechanism increased the plastic demand in the base section and caused the fracture of the web reinforcement. This, in turn, affected the strength to lateral load, and the spalling, buckling of the longitudinal reinforcement of the ends, and crushing of the concrete were accelerated. Table 5 shows that the damage progresses similarly for the walls with electro-welded mesh in the web and that its behavior differs moderately from that exhibited by the M2T wall with ductile reinforcement in the web and significantly concerning the M4I wall with special BEs. In Fig. 8, the state final of the walls is shown.

A brief description of the damage evolution and failure mode of each of the walls is presented below:

M2T: From a drift of 0.4%, the cracking was distributed over the entire height of the wall (Fig. 7a and b). As the displacement increased to 0.6% drift, a crack was detected at the foundation-wall interface that widened to cause detachment of the base of the wall. This gave rise to

rigid body rotation from 0.8% drift. The maximum strength in the positive direction was reached for a drift of 1.0%. At the first cycle of 1.2% drift, fracture of the web reinforcement at both ends was initiated. For the second cycle, damage at the compressed ends extended to a height of 220 mm. For the 1.4% drift, the maximum strength was reached in the negative direction, fracture of the fifth web bar occurred, and concrete spalling in the flange began. The strength continued to decrease in both directions due to fracture of the reinforcement and progressive spalling that exposed the flange reinforcement and caused buckling of one of its bars for the 1.6% drift. The failure occurred for the next drift level, 1.8%. In the last cycles, reinforcement buckling was observed at both ends, with greater intensity in the flange, and the remaining web bars fractured.

M3R: From a drift of 0.2%, the cracking was distributed over the entire height of the wall (Fig. 7c and d). The cracking at the interface initiated for drifts of 0.4 and 0.6% in both directions, respectively. The growth of this crack caused the detachment of the base and originated rigid body rotations from the drift of 1.0%. The maximum strength in the positive and negative directions was reached for drifts of 0.8% and 1.0%, respectively. For a drift of 1.0%, fracture of the web reinforcement started. As the displacements increased to a drift of 1.2%, the damage concentrated in the compressed zones to an average height of 225 mm, and the entire electro-welded reinforcement fractured. In the second cycle, spalling of the concrete began at both ends. In the first cycle of the 1.4% drift, the east end reinforcement buckled, and in the second cycle, the concrete crushed, leading to failure in the positive direction as the strength was reduced by about 80%. For the next drift level, a similarly occurred at the west end, and failure occurred for a drift of 1.6%.

M4I: The cracking was distributed over the entire height of the wall, starting at a drift of 0.4% (Fig. 7e and f). The interface crack was detected for a drift of 0.8%. This crack widened considerably from drifts of 1.4 and 1.0% in both directions, respectively, which initiated rigid body rotation. By 1.2% drift, spalling of the concrete at both ends and fracture of the web reinforcement began. Upon reaching a drift of 1.4%, crushing of the concrete cover at the east BE was observed, and the maximum strength in the positive direction was reached. For a drift of 1.8%, the last web wire fractured, and spalling of the web concrete was observed. At the next drift level, 2.0%, maximum strength was reached for the negative direction, and buckling of the east BE reinforcement was observed. Progressive spalling and crushing of the concrete in the bottom zone of the BEs caused failure for a drift of 2.5%.

M5T: Interface cracking was detected from a drift of 0.2%. The cracking was distributed up to a height of approximately 2000 mm (Fig. 7g and h). The onset of detachment from the base and peak strength in both directions occurred for the first cycle of 0.6% drift. For the second cycle, the first web wire fractured. In the first cycle of the next drift level, 0.8%, seven web wires fractured. This caused that in the

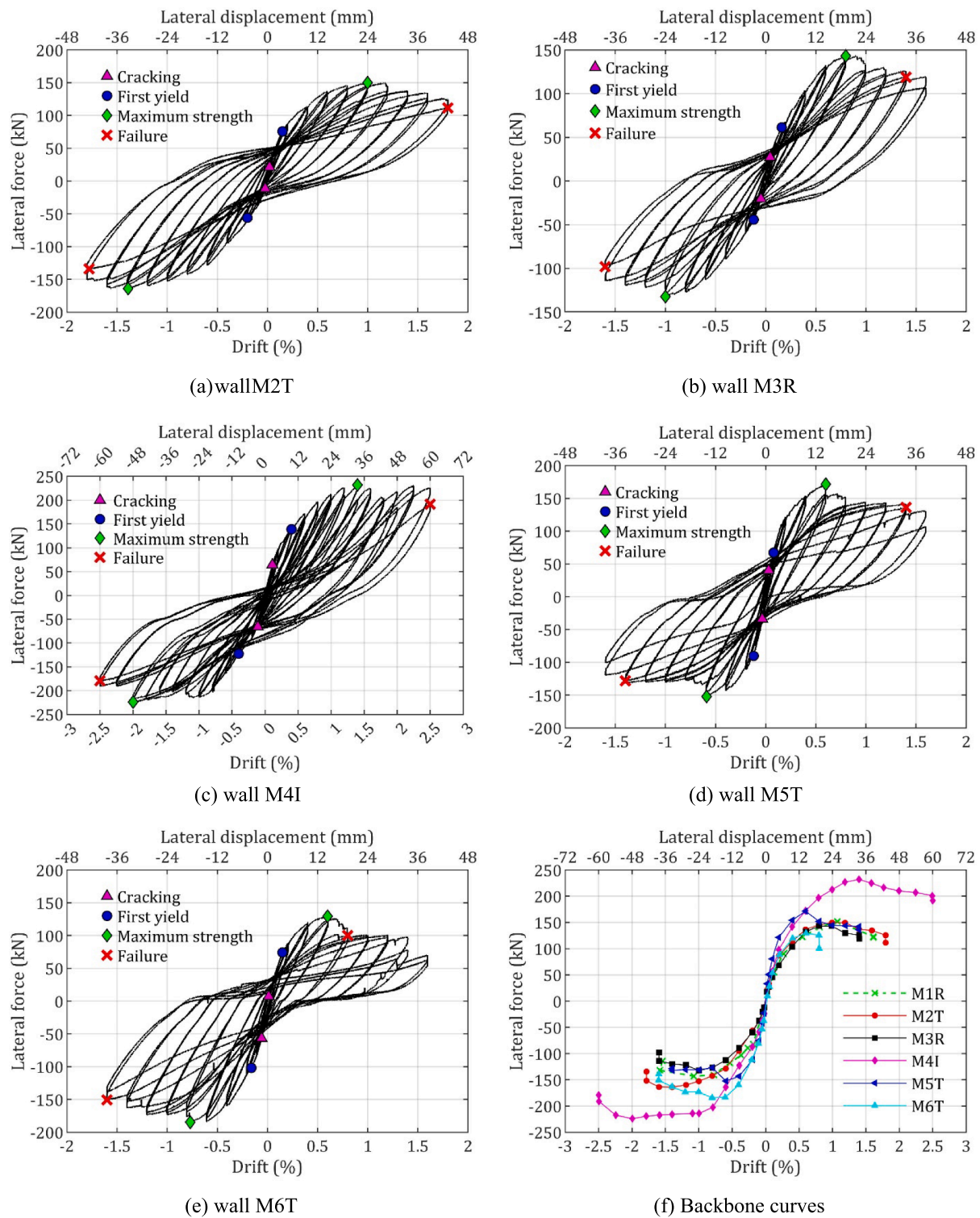


Fig. 6. Force-drift relationship of the walls, and backbone curves.

second cycle strength to decrease by 18.6 and 16.8%, concerning the maximum resistance, in the positive and negative directions, respectively. In the first cycle of the 1.0% drift, a crack appeared in the flange that reached a thickness of 4 mm and caused it to fracture at a height of 180 mm. Concrete spalling started in the affected area of the flange and subsequently at the other end for the 1.2% drift. The wall reached failure

in both directions at 1.4% drift due to buckling of the end reinforcement and crushing of the concrete.

M6T: Interface cracking was detected for a drift of 0.1%. The cracking was distributed up to a height of 2200 mm (Fig. 7i and j). The onset of detachment from the base and maximum strength in the positive direction occurred for the first cycle of the 0.6% drift. A significant

Table 4

Drift and normalized shear stress for limit states.

Limit state	I Cracking		II First yield		III Maximum resistance		IV Ultimate resistance	
Wall	Δ (%)	$\tau/\sqrt{f_c}$	Δ (%)	$\tau/\sqrt{f_c}$	Δ (%)	$\tau/\sqrt{f_c}$	Δ (%)	$\tau/\sqrt{f_c}$
Positive direction (end east subjected compression)								
M2T	0.021	0.03	0.15	0.10	1.00	0.20	1.80	0.15
M3R	0.045	0.04	0.16	0.08	0.80	0.19	1.40	0.16
M4I	0.110	0.09	0.40	0.19	1.40	0.31	2.50	0.26
M5T	0.030	0.05	0.08	0.08	0.60	0.20	1.40	0.16
M6T	0.013	0.01	0.15	0.09	0.60	0.15	0.80	0.12
Negative direction (end west subjected compression)								
M2T	-0.022	0.02	-0.20	0.08	-1.39	0.22	-1.78	0.18
M3R	-0.047	0.03	-0.12	0.06	-1.00	0.18	-1.60	0.13
M4I	-0.110	0.09	-0.40	0.16	-2.00	0.30	-2.50	0.24
M5T	-0.034	0.04	-0.12	0.11	-0.59	0.18	-1.40	0.15
M6T	-0.058	0.07	-0.16	0.12	-0.77	0.21	-1.60	0.18

Table 5

Damage progress recorded in the tests.

Description of the event	Drift level (%)									
	M2T	M3R	M4I	M5T	M6T					
Onset of cracking [I]	0.025 ^a	+0.021 ^b	0.05	+0.045	0.2	+0.11	0.05	+0.03	0.025	+0.013
		-0.022		-0.047		-0.11		-0.034	0.1	+0.058
First yield of reinforcement [II]	0.2	+0.15	0.2	+0.16	0.4	+0.40	0.1	+0.077	0.2	+0.15
		-0.20		-0.12		-0.40	0.2	-0.124		-0.165
Significant increase in cracking	0.4	+0.40	0.2	+0.20	0.4	+0.40	0.4	+0.40	0.4	+0.40
		-0.40		-0.20		-0.40		-0.40		-0.40
Onset of cracking at the foundation-wall interface	0.6	+0.60	0.4	+0.35	0.8	+0.80	0.2	+0.20	0.2	+0.20
		-0.60	0.6	-0.45		-0.60		-0.20	0.1	-0.10
Onset of detachment from the base	0.8	+0.79	1.0	+0.80	1.4	+1.30	0.6	+0.55	0.6	+0.60
		-0.80		-0.90	1.0	-0.90		-0.55		-0.60
Maximum resistance - [III]	1.0	+1.00	0.8	+0.80	1.4	+1.40	0.6	+0.60	0.6	+0.60
		-1.39	1.0	-1.00	2.0	-2.00		-0.59	0.8	-0.77
Onset of the web reinforcement fracture	1.2	+1.18	1.0	+0.91	1.6	+1.60	0.6	+0.48	0.8	+0.69
		-1.16		-0.92	1.2	-1.10	0.8	-0.49		-0.77
Onset of concrete spalling	1.4	N.R.	1.2	+0.40	1.2	+1.2	1.2	+1.20	1.2	+1.0
		-1.06		N.R.		-1.2		N.R.		-0.58
Onset of buckling of ends reinforcement	1.6	N.R.	1.4	+1.40	-	-	1.4	+1.40	1.4	N.R.
		-0.60	1.6	-1.20	2.0	N.R.		N.R.	1.2	-0.60
Onset of concrete crushing	1.6	N.R.	1.4	N.R.	N.R.	N.R.	1.4	N.R.	1.4	N.R.
		N.R.		N.R.	1.4	-0.67		N.R.		N.R.
Wall Failure [IV]	1.8	+1.80	1.4	+1.40	2.5	+2.50	1.4	+1.40	0.8	+0.80
		-1.78	1.6	-1.60		-2.50		-1.40	1.6	-1.60

(a) The first column indicates the drift level of the displacement protocol in which the event or damage was detected.

(b) The second column indicates the direction and the drift just at the instant the event or damage occurred.

N.R. No record was obtained of the moment in which the damage occurred.

crack also emerged in the flange at a height of 330 mm. During the first 0.8% drift cycle, three web wires fractured, reducing the strength in the positive direction by 14.1%. In the other direction, the maximum strength was reached, and three other wires were immediately fractured. In the second cycle, the positive strength decreased drastically to 71%, owing to the fracture of more wires. In this case, the premature and progressive fracture of the electro-welded mesh had a more significant impact on reducing strength than observed in the M5T wall and caused considerable signs of damage, which led to the failure of the wall in the positive direction. At a drift level of 1.0%, the flange fractured, and a significant vertical crack was observed on the east lateral face, which emerged from the base and ascended to a height of 200 mm. At the next drift level of 1.2%, concrete spalling at both ends and buckling of the reinforcement in the flange were visually detected. The buckling of the bars at the other end became noticeable at a drift of 1.4%. For the next drift of 1.6%, the buckled bar of the flange fractured, and failure occurred in the negative direction.

In the M2T wall, the onset of cracking at the interface, the fracture of the web reinforcement, and the maximum strength were reached for drifts higher than those in the walls using electro-welded mesh. This is

explained by the use of steel of higher ductility and with a higher strain hardening ratio. In this wall, the hysteresis cycles were more stable, and a greater displacement capacity was achieved. Although the web reinforcement (#2 bars) has a deformation capacity four times higher than that of the electro-welded steel, it is also fractured because the T-cross section implies a considerably higher tensile strain demand than that of a rectangular wall, such as the M3R. Specifically, for the 1.2% drift at which the web reinforcement fracture initiated in the M2T, the strain demand could have been more than double, on average, than that in the M3R when the drift was 0.9%, which is the instant at which the mesh fracture initiated for this wall (Table 5). In addition, the M2T wall had a wider interface crack than the M3R, which was approximately twice as large for the aforementioned drifts (Fig. 12), which also increased the strain demand of the web reinforcement. For the M4I wall, the confined BEs significantly improved their performance. The cracking pattern was different from that of the other walls. The confinement maintained the integrity of the concrete core and delayed the initiation of the fracture of the reinforcement and the buckling of the longitudinal reinforcement of the ends. This allowed the wall to maintain its capacity up to a drift 50% greater than the failure drift of walls of the same length.

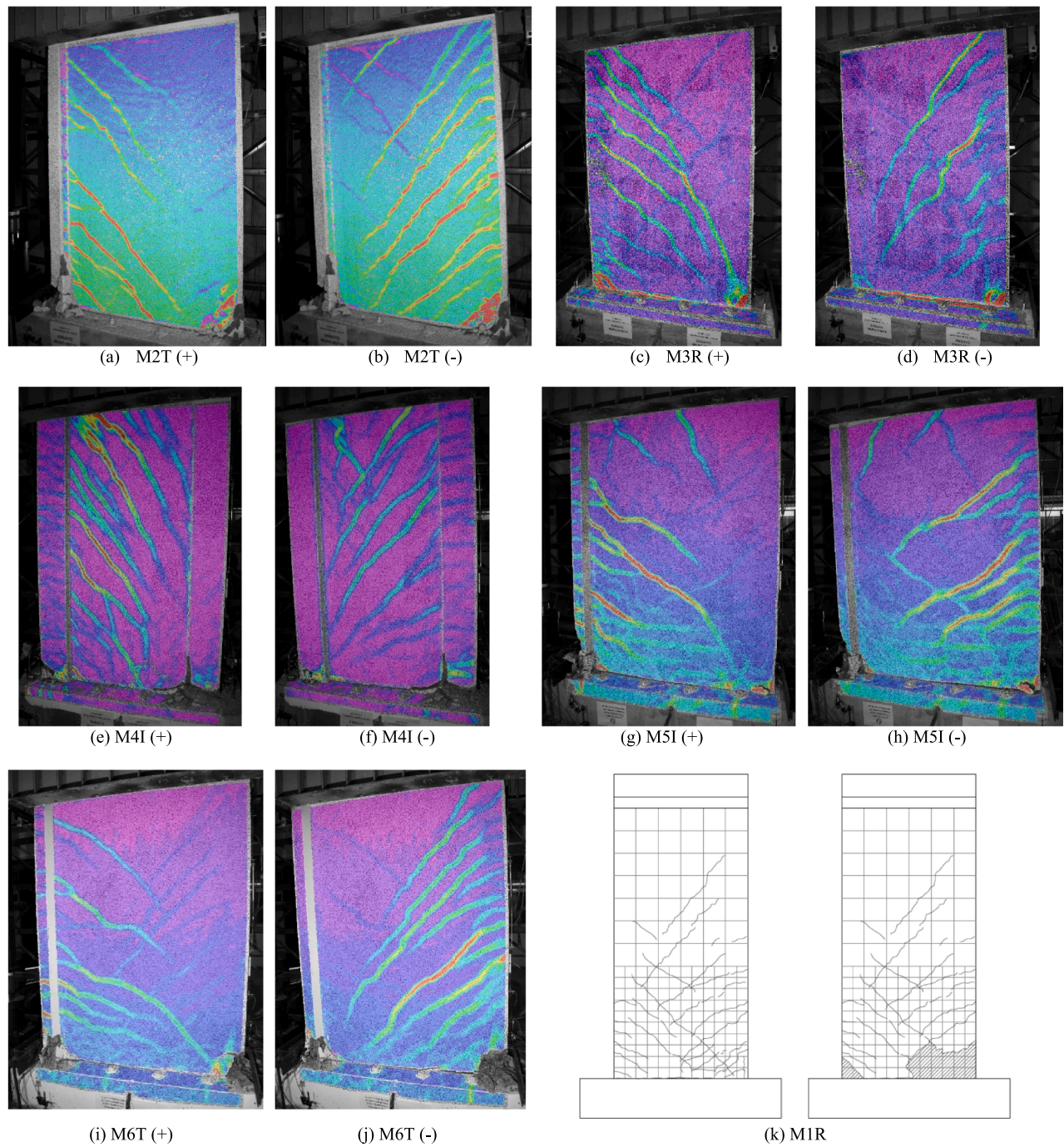


Fig. 7. Cracking pattern of test walls.

The separation of the transverse reinforcement from the ends of the M3R wall influenced the cracking pattern. At the west end, where crossies spaced every 75 mm were used (Fig. 1), a greater number of cracks were formed, but they were of less thickness and length than those formed at the other end, whose crossies were spaced every 150 mm. This condition indicated that both the extension of the damage and the buckling length of the reinforcement were greater in the east end, as shown in Fig. 8f. Additionally, at that east end, the lap splices of the longitudinal reinforcement were arranged just from the interface, unlike the other end, whose lap splices were located above the expected plastic hinge zone.

In the M4I wall, the east BE was more affected than the other end. This was due to the asymmetric arrangement of longitudinal

reinforcement and to the fact that, despite having the same amount of transverse reinforcement, both the relationship between the spacing of the hoops and the diameter of the longitudinal reinforcement (S/d_b) as the distance between the branches of the hoops (h_x) were greater for the east end (i.e., S/d_b : 4.7 vs. 3.1, and # of branches: 2 vs. 3). In Fig. 7e and f, the difference between the cracking pattern of both ends is observed, with cracks less spaced but of smaller amplitude for the west BE that had better confinement conditions. This wall was also subjected to a drift of 2.75% after failure and exhibited fracture of an internal bar of the reinforcement of the EB east and crushing of a sector of the web adjacent to this same end. This phenomenon usually occurs in elements with large flanges and thin webs subjected to high shear stresses [28]. The hoops did not show signs of involvement.

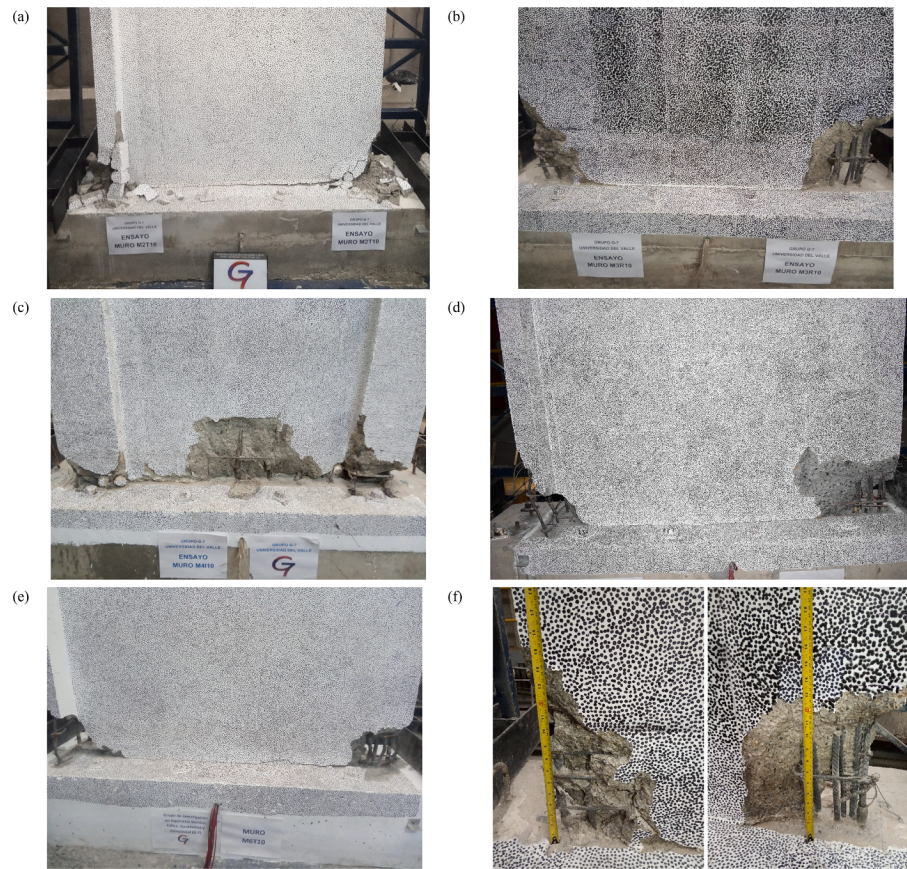


Fig. 8. Final state of the walls, a) M2T, b) M3R, c) M4I, d) M5T, e) M6T, f) Detail of damage to wall M3R.

4. Analysis and discussion of results

4.1. Strength and capacity of displacement

The shear stress values for the maximum strength limit state varied between 0.15 and 0.31 times $\sqrt{f_c}$ (MPa) for the M6T and M4I walls, respectively. The average maximum shear stress, not including the M4I wall, was $0.19 \sqrt{f_c}$. This is only 12% greater than the theoretical shear strength provided by concrete, calculated as $0.17 \sqrt{f_c}$ MPa [23]. These values confirm that the shear effect did not have a predominant influence on the progress of the damage and the displacement capacity of the walls.

The M2T and M6T walls with more longitudinal reinforcement at the web end than at the flange reached higher strength in the negative direction. The M3R wall achieved almost the same capacity in both directions, with a difference in strength of only 8%. Wall M4I, despite having more reinforcement in the west BE, achieved almost the same strength level in both directions. The M5T wall, with a similar amount of reinforcement at both ends, exhibited a higher capacity in the positive direction. For the M6T wall, a higher strength in the positive direction (similar to that of M5T) was expected, but the fracture of the mesh wires caused a strength drop of more than 20%. In addition, a subsequent test of the steel used in the flange of this wall showed that the yield stress was 15% lower than the average yield stress of the reinforcing steel of the other walls. The backbone curves in Fig. 6f show that the M4I wall exhibited the highest capacity in strength and displacement of all the walls, as expected, and that the 1200 mm long M1R wall achieved almost the same strength as the 1500 mm long M2T and M3R walls, due to the effect of the higher axial load and shear span ratio.

The M3R and M6T walls were the only ones that failed for a different value of drift in each direction. The walls reinforced with electro-welded mesh (M1R, M3R, M5T, y M6T) reached ultimate drifts up to 1.4 and

1.6%. However, the M6T wall failed in the positive direction at a drift of 0.8%, and for the M5T wall, the failure could have been defined for this same drift level due to the high loss of resistance caused for the fracture of the web reinforcement. This indicates that, although this type of walls could achieve a response moderately ductile because they are short walls (i.e., $l_w \leq 2000$ mm) and with a considerable amount of longitudinal reinforcement at the ends, they could also fail for drifts lower or close to 1%. It is important to note that the failure of the M6T wall, in the positive direction, not only obeyed the criterion of loss of strength of more than 20%, but for that level of drift of 0.8%, although visibly no spalling or crushing of the concrete was observed, several factors indicated the presence of significant damage. First, for that drift, the fracture of 8 of 9 web wires was detected. Second, the compressive strain in the east-end fiber reached a value of 0.0036, suggesting even higher compression at the base level. This was confirmed at the next drift of 1.0%, where a vertical crack emerged at the base of the east lateral face that ascended to a height of 200 mm. In addition, at the west end, the fracturing of the flange was initiated.

Wall M4I achieved the highest drift of all the walls, 2.5%. The analysis of its response indicates that the configuration of its boundary elements originated the increase of its deformation capacity and the delay of the onset of yielding, cracking, and damage in general, in comparison with the walls with electro-welded mesh. In particular, it was found that increasing the cross-section of the ends, with a thickness that ensured effective confinement of the concrete, had a high impact on the deformation capacity. Also, the increase in the total amount of longitudinal reinforcement in the EBs favored a greater distribution of damage along the wall height concerning the other units, as shown in Fig. 7, and discussed in the section on deformation components.

The M2T wall, with ductile reinforcement in the web, showed the highest displacement capacity (1.8%) after M4I. For this wall M2T, a softer resistance degradation was observed concerning the walls

reinforced with electro-welded mesh, for which, from the onset of the fracture of the reinforcement in the web, a more marked resistance drop was observed.

The failure mode and the displacement capacity of the test walls can be explained according to the following reasons:

1. The low strain-hardening ratio (f_{max}/f_y) of the electro-welded mesh and the low amount of longitudinal reinforcement in the web caused the plasticity demand to be concentrated in a main crack in the foundation-wall interface to early drifts [3,10,22]. In the case of the M2T wall, it is hypothesized that the low amount of ductile steel in the web generated the same effect of concentration of plasticity, but this occurred for a higher level of drift due to the higher strain-hardening ratio of bars # 2.
2. As the displacement increases, the interface crack is prolonged toward the ends and the center of the wall, and its thickness is considerably widened. This causes great demands on strain at the base of the wall, which causes premature fracture of the electro-welded reinforcement that has a limited ductility, $\epsilon_u \approx 2.5\%$, [10]. The same occurs for the M2T wall for higher levels of drift.
3. The progressive fracture of the web reinforcement affects the resistance of the wall and can cause an abrupt and premature reduction of even more than 20%. Fracturing this reinforcement, which increases the strains in the end bars, accelerates the damage, that is, the spalling, buckling of the longitudinal reinforcement, and crushing of the concrete, which leads to the failure of the wall. For the M2T and M4I walls, the effect of reinforcement fracture on the strength appeared to be less critical.

Despite the foregoing, the reinforcement concentrated at the ends, subjected to high demands on strain and to the effect of the reinforcement sliding within the foundation, favored the formation of secondary cracks along the height of the wall [29]. This caused the rotation of rigid body to be not the only source of deformation, and that the inelastic displacement capacity of the walls was increased.

4.2. Stiffness degradation

The degradation of the normalized stiffness, calculated as the relationship between the stiffness for each cycle or level of drift (K_i) and the initial stiffness of the wall (K_0), indicates the progress of the concrete cracking for each increase in displacement demand. Fig. 9a and b show the stiffness degradation for the positive and negative directions. The stiffness K_i was calculated as the average of the stiffnesses of the two cycles of each level of drift. Drifts of 0.5%, 1%, and 1.43% are also indicated for reference. The last two values correspond to the limit drifts

established according to the NSR-10 code for wall buildings designed with uncracked and cracked sections, respectively. Both figures include the stiffness degradation of the M1R wall, although it was subjected to a different shear span ratio and to a displacement protocol starting from a drift of 0.15%. The stiffness degradation was similar for all units, although slightly less for the M1R wall. When a drift level of 0.5% was reached, the stiffness was reduced on average to 30% of the initial stiffness. For a drift of 1%, the stiffness was reduced on average to 17%, and for a drift of 1.43%, the reduction was up to 11%. These degradation levels are congruent with what has been reported and recommended by Blandón et al. [3,4] regarding the convenience of using cracked sections for design purposes with a considerable level of cracking.

4.3. Dissipated energy

The dissipated energy was determined as the area under the curve of the hysteresis cycles of the force–displacement response of each wall. Fig. 10a and b show the dissipated energy and the accumulated dissipated energy as a function of drift, respectively. In both cases, the energy dissipated until the end of each test was calculated, and the M1R wall was included as a reference. The 1500 mm long M2T, M3R, and M4I walls exhibited almost the same amount of energy dissipated down to a drift of 1.2%. The M5T and M6T walls, which were 1700 mm long and whose difference consisted only of the amount of longitudinal reinforcement at the web-end (east), also dissipated approximately the same amount of energy up to a drift of 1.2%. These units dissipated, on average, 61% more energy than the shorter walls for a drift of 1.0%. The least amount of energy dissipated was that of the shortest wall, the M1R, of 1200 mm. According to its displacement capacity, the M4I wall with confined EBs exhibited a greater energy dissipation; however, the M2T wall, with ductile reinforcement in the web, dissipated more energy in all cycles than the walls of the same length with electro-welded reinforcement, including the M4I. This corresponds to the hysterical response and damage progress observed in this wall, in which the effect of the web reinforcement fracture was less critical and occurred at higher levels of drift.

4.4. Deformation components

Fig. 11 shows the contributions of the bending and shear deformation, slip, and rigid body rotation components for each wall, calculated as the average in each direction and for both cycles. In addition, indicates the drift estimated as the onset of rotation around the neutral axis. The sum of these contributions was acceptably matched with the value of the total displacement measured directly at the top of the wall. The distribution of the deformation components was relatively similar in

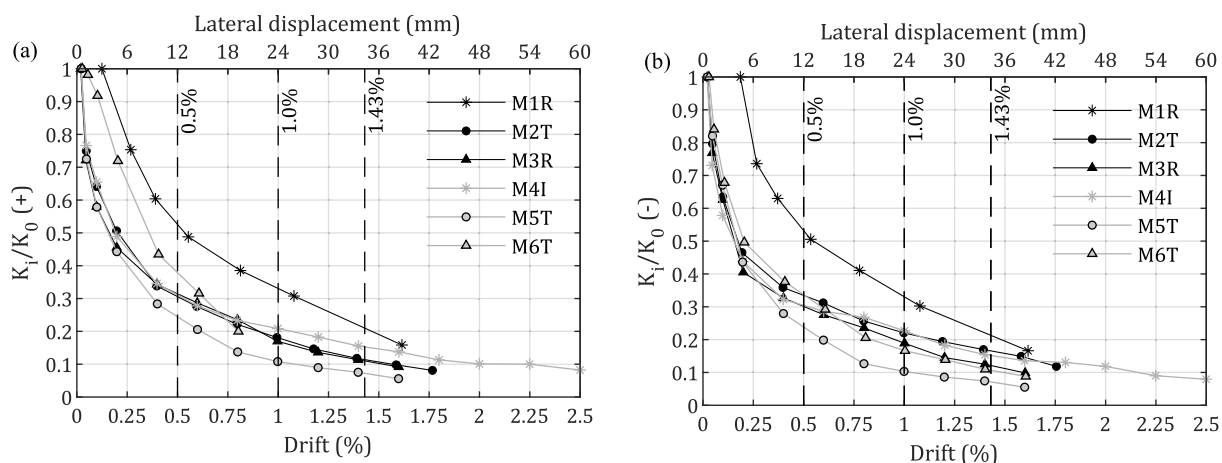


Fig. 9. Stiffness degradation, a) positive direction and b) negative direction.

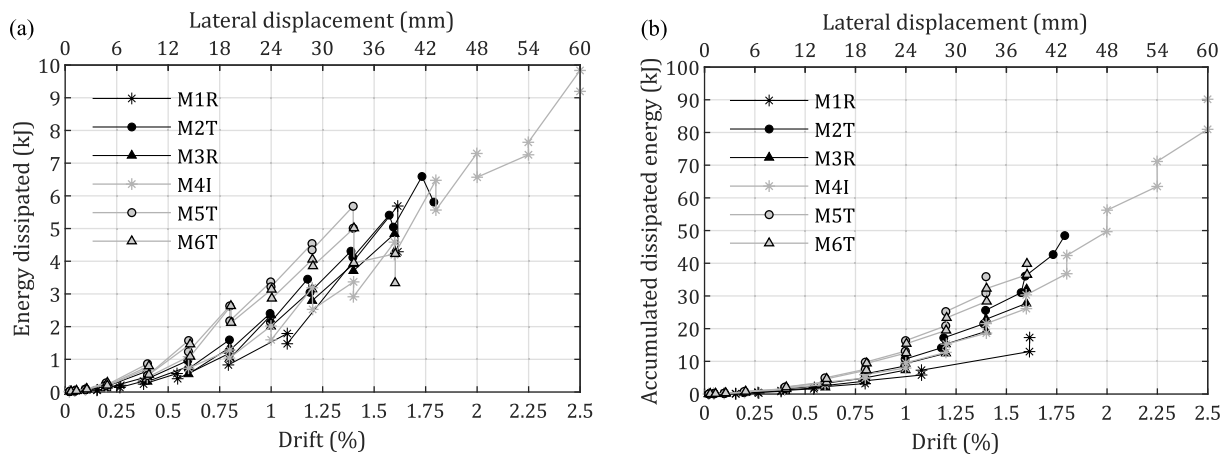


Fig. 10. a) Dissipated energy and b) accumulated dissipated energy.

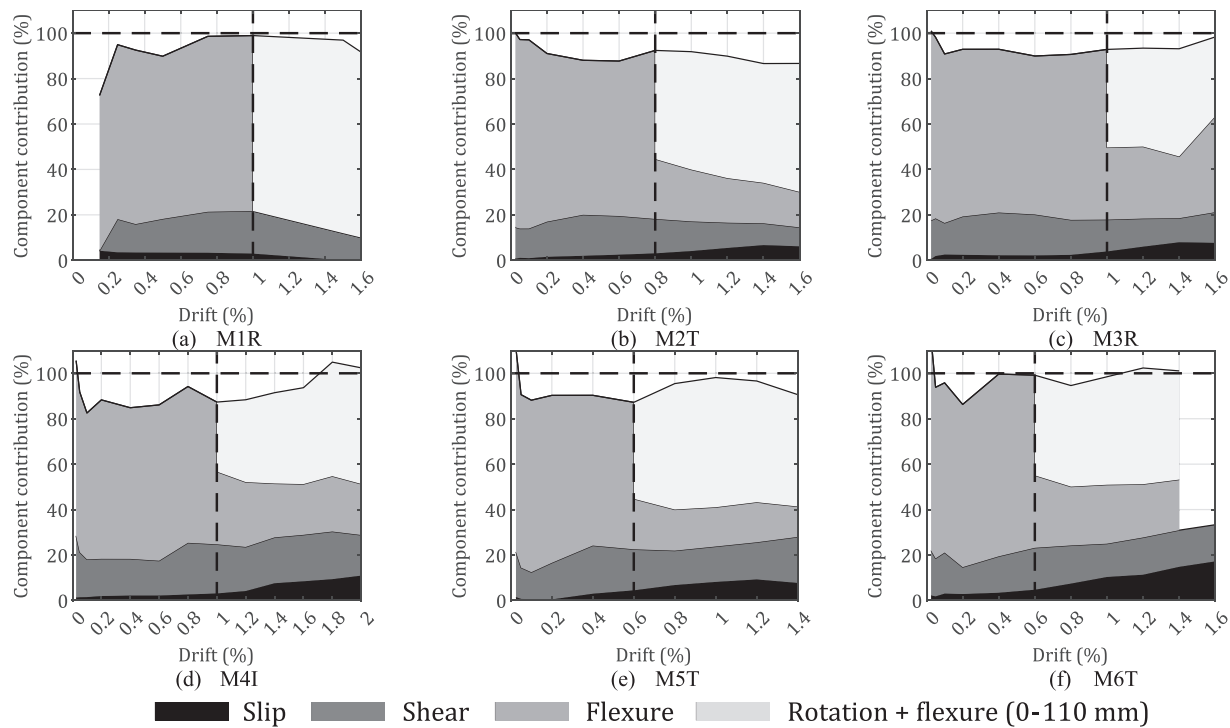


Fig. 11. Deformation components.

both directions. The total displacement of each wall was corrected using the deformation associated with the rotation and sliding of the foundation. The shear deformations were calculated according to Hiraishi [30] from the sensors located at the bottom diagonal (Fig. 4a). The bending deformations due to cracking propagation in height were calculated by the double integration of the curvature obtained from the vertical displacements recorded by the sensors installed at both ends. In both cases, deformations were also calculated with data obtained by photogrammetry (DIC). The slip was calculated from the records of the sensors located at the base of the wall. Because it was not possible to separate the effect of the interface crack from the bending deformations in the lower section of the wall, covered by the first sensors (T1-01 and T1-02) located at a height of 110 mm (Fig. 4a), the bending component was defined in two stages, before and after the instant (drift) defined as the onset of rigid body rotation. Before this drift, the deformation component corresponds to the bending in the whole wall, and after the beginning of the rotation, this component is divided into i) bending

deformations measured from the first sensor (from 110 to 2400 mm) and ii) rigid body rotation plus the bending of the first 110 mm of wall height. Fig. 11 includes wall M1R, for which it was not possible to separate the effect of bending from the rigid body rotation.

The registration of the sensors installed on the lateral faces of the walls and those used for out-of-plane measurements were used to validate the displacements obtained by image processing (DIC) along the ends of the walls every 50 mm. It is worth clarifying that although the sensors used to measure shear deformations were located at the base of the wall, thus excluding shear deformations occurring at the top, the calculations were complemented with data obtained by photogrammetry along the height of the wall, which confirmed that these deformations are more predominant at the bottom.

The distribution of the deformation components had a similar trend for all the walls. Sliding at the base of the wall averaged 4.3%. The shear deformation contribution averaged 16.3%, with a variation between 13.4 and 19.6%, the latter value corresponding to wall M4I, which had

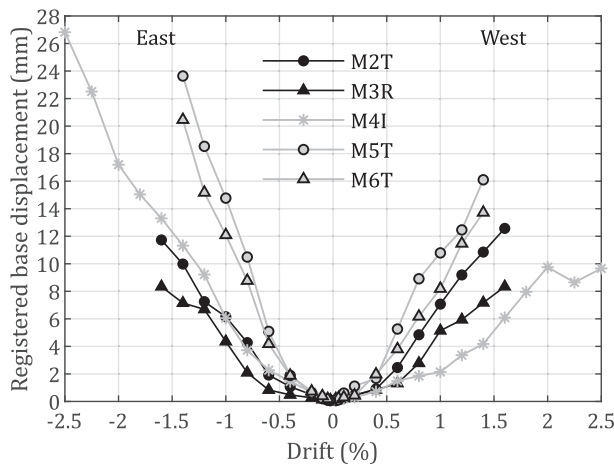


Fig. 12. Progress of interface crack and flexural cracking in the lower span (0–110 mm), monitored by the T1-01 and T1-02 sensors.

shear stress 1.6 times higher than the other walls for the maximum strength limit state. The bending component, before the onset of rigid body rotation, averaged 74.6%, with a variation between 68.6 and 78.5%. This is approximately 4.5 times greater than the shear effect. After rigid body rotation, the bending component at the top of the wall (110–2400 mm) was approximately 23.6%, with a variation between 16.6 and 33.7%. This confirms that the cracking distribution above the interface's crack was considerable. The contribution of rotation plus bending at the bottom of the wall (0–110 mm) averaged 48.2%, with a

variation between 41.9 and 53.8%. According to the cracking pattern exhibited, corresponding to the formation of a main crack at the interface and the presence of a considerable number of secondary cracks distributed in height, flexural deformation and rotation contributed approximately 72% of the total displacement of the walls.

Fig. 12 shows the displacement recorded by the sensors (T1-01 and T1-02) located 110 mm from the base on the lateral faces of the walls (Fig. 4a). This displacement indicates the progress of interface crack and flexural cracking in the lower span (0–110 mm). The longest walls, M5T and M6T, exhibited the largest cracks, and, therefore, their rigid body rotations were among the highest. The M3R wall exhibited the smallest crack, i.e., the smallest rigid body rotation and largest flexural component above the base. The M6T wall, with higher reinforcement content at the web end than M5T, exhibited a smaller crack than M5T and, thus, a lower average rigid body rotation (47.8 vs. 53.8%), and higher bending deformations above the interface (24.4 vs. 16.6%). Whereas the M2T wall with higher reinforcement content at the web end than the M3R exhibited a larger crack than this, and, therefore, a higher rotation (53.7 vs. 41.9%) and lower bending component above the interface (19 vs. 33.7%). This is likely because the bars in the M2T wall had a larger diameter, which would cause greater sliding of the reinforcement within the foundation. The behavior of the M4I wall was different in each direction. The east end, with a smaller amount of longitudinal reinforcement than the other end, developed a larger crack. This wall showed one of the lowest contributions of rotation plus bending at the base (44%), consistent with its higher amount of reinforcement compared to the other units.

A comparison with walls W1 to W4, tested by Blandón et al. [3], of similar characteristics to those of this investigation, indicated that in

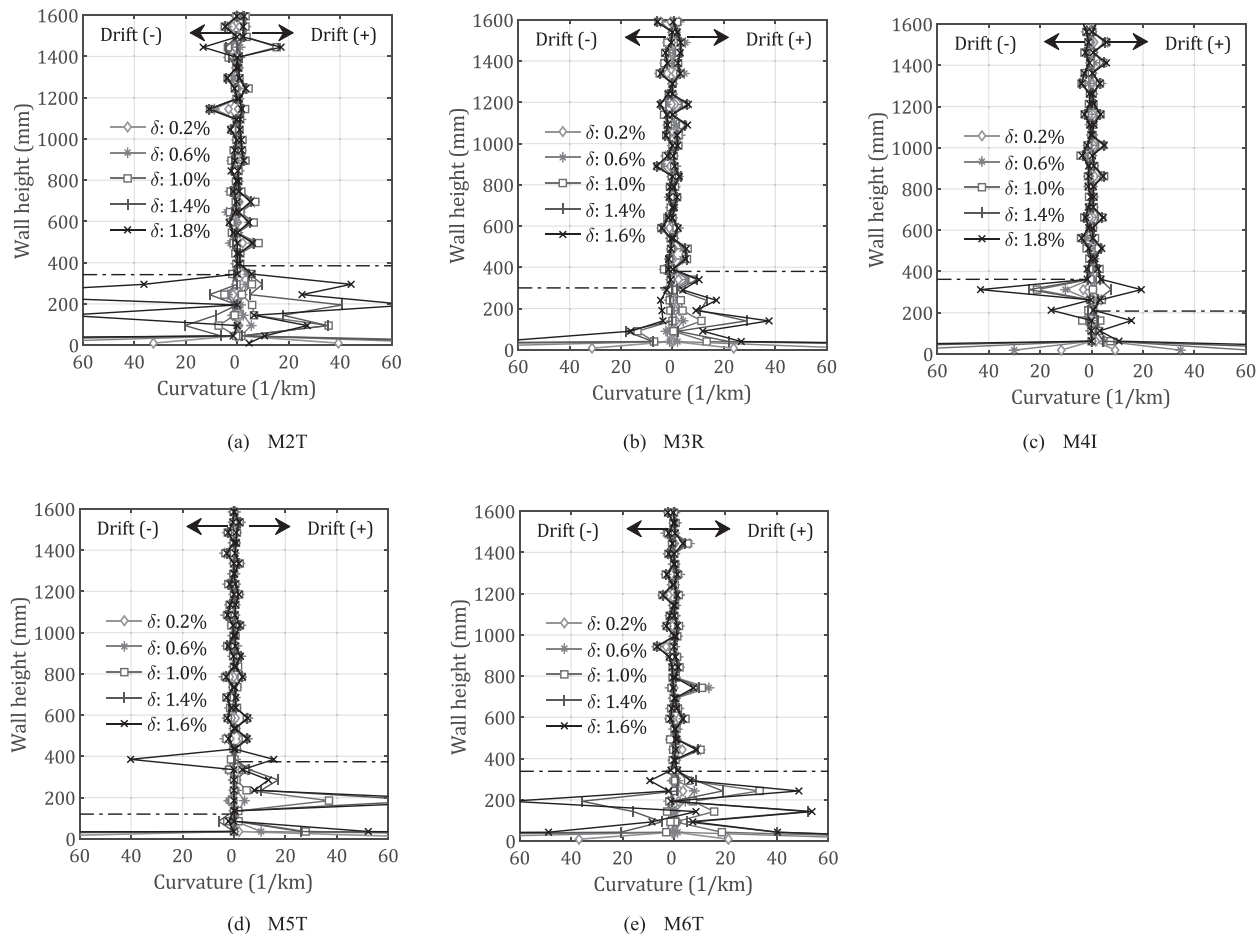


Fig. 13. Average curvature distribution.

Table 6
Plastic zone height L_{pz} and damage extent.

Wall	L_{pz}		L_{pz}		Damage extent		Max. strain in bars a 300 mm (W), and Drift (%)		Max. strain in bars a 300 mm (E), and Drift (%)	
	(W) curvature	(E) curvature	(W) strain	(E) strain	(W)	(E)	Tensile	Compressive	Tensile	Compressive
M2T	385	342	375	341	320	380	0.012 (1.2) ^a	0.0013 (1.0)	0.0049 (1.2)	0.0038 (1.2)
M3R	380	300	380	325	380	340	0.016 (1.2)	N.R.	0.0048 (1.4)	0.0015 (0.6)
M4I	208	362	165	360	200	500	0.006 (2.5)	0.0028 (1.8)	0.0091 (0.6)	0.0014 (0.5)
M5T	374	120	371	121	400	300	0.012 (1.2)	0.0025 (1.0)	0.0066 (1.4)	0.0014 (1.2)
M6T	339	339	341	387	430	300	0.007 (0.6)	0.0011 (0.4)	0.014 (1.0)	0.0031 (1.2)

(a) The value in parentheses indicates the drift at which the maximum strain was recorded.

relation to the contribution of shear deformations, two of their walls showed a minimum contribution to the total deformation, and the other two exhibited a contribution of 25.9% and 22.9% (W6 and W7). This contribution was higher than that of the walls in this study, which for the same maximum resistance limit state, reached values between 11.3 and 19%. In the bending component, these authors reported a contribution of 42.8, 0, 24.6 and 27.7% for walls W1 to W4, respectively, while in this investigation, higher values were recorded, between 62.6 and 76%, just before the onset of rigid body rotation, this is, close to the maximum resistance limit state. The contribution of rigid body rotation for walls W1 to W4 (42.8, 64.5, 44.1 and 37.3%, respectively) was slightly higher for the same limit state than for walls M1R to M6T, whose rotation (including the base bending deformation component) was between 36.3 and 55.4%. The higher contribution of bending and apparent lower influence of the rigid body rotation effect could be explained by the fact that the test walls were shorter in length and had more longitudinal reinforcement at the ends than walls W1 to W4.

4.5. Curvature distribution

The average curvature distributions were estimated by recording the sensors installed on the lateral faces of the walls and with photogrammetry (DIC) on the south face. Both methods presented an acceptable correlation. However, a considerably shorter length gauge (50 mm) was used with the images, resulting in a better estimation. That is, the vertical displacements were obtained every 50 mm, in height, near the extreme fiber or edge of the walls. From these values, the curvature was calculated. Fig. 13 shows the profiles obtained with this technique for the first cycle of the drifts of 0.2, 0.6, 1.0, and 1.4%, as well as the last drift in which it was possible to take a reliable record. According to the reinforcement content used at the ends of the walls, which allowed the formation of a significant number of secondary cracks along almost the entire height (Fig. 7), several curvature peaks were present at the location of cracks of considerable thickness, which is observed in the profiles obtained. The initial curvature at the base of the wall includes the effect of the strain penetration of the longitudinal reinforcement. The curvature distribution corresponds to the observed cracking patterns and behavior. The maximum average curvatures were located at the bottom of the walls, confirming the effect of rigid body rotation and the bending deformation component at the base of the walls (Fig. 11). For the initial drifts, the curvature distribution was similar in all units, and from drifts greater than 0.4%, the curvature peaks at the height of the cracks were more noticeable.

The height of the plastic zone L_{pz} , which corresponds to the height of the wall over which inelastic deformations take place, was estimated from the curvature profiles. L_{pz} was calculated as the height above the base for which the curvature was equal to the yield curvature [10]. The value of the latter was estimated, for all walls, as $\phi_y = 1.9 \text{ km}^{-1}$ [31] from the analysis of sections. Fig. 13 indicates the location of the plastic zone at each end with a dashed line. The region in which plastic deformation was concentrated varied from one wall to another. L_{pz} ranged between 120 and 385 mm (Table 6). Wall M2T exhibited the largest inelastic deformation concentration zone, while wall M4I, despite having a considerable cracking distribution in height, and a higher displacement capacity, showed a smaller L_{pz} than M2T at the west end, according to this method. However, as described below, tensile strains measured in the longitudinal reinforcement did indeed confirm a greater extent of plasticity in M4I. Similarly, in wall M5T, a reduced web-end deformation concentration was observed. Walls M3R and M6T exhibited a plastic zone height between 3 and 4 times the wall thickness (t_w). Therefore, a reduced plastic hinge length would be expected. In the tests of Blandón et al. [4,21] of walls with lower longitudinal reinforcement amounts and less secondary crack formation, values close to 250 mm for the plastic zone were observed.

The height of the plastic zone was also estimated from the distribution of tensile strains along the height of the walls. L_{pz} was determined as

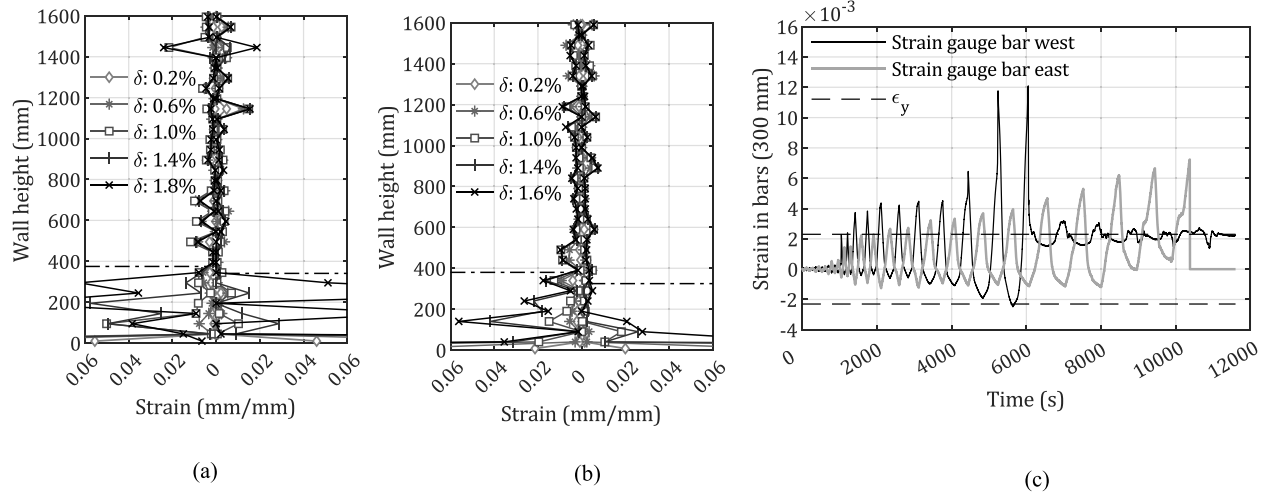


Fig. 14. a), b) Tensile strain profiles for walls M2T and M3R, and c) strains measured in the external bars of wall M5T at a height of 300 mm.

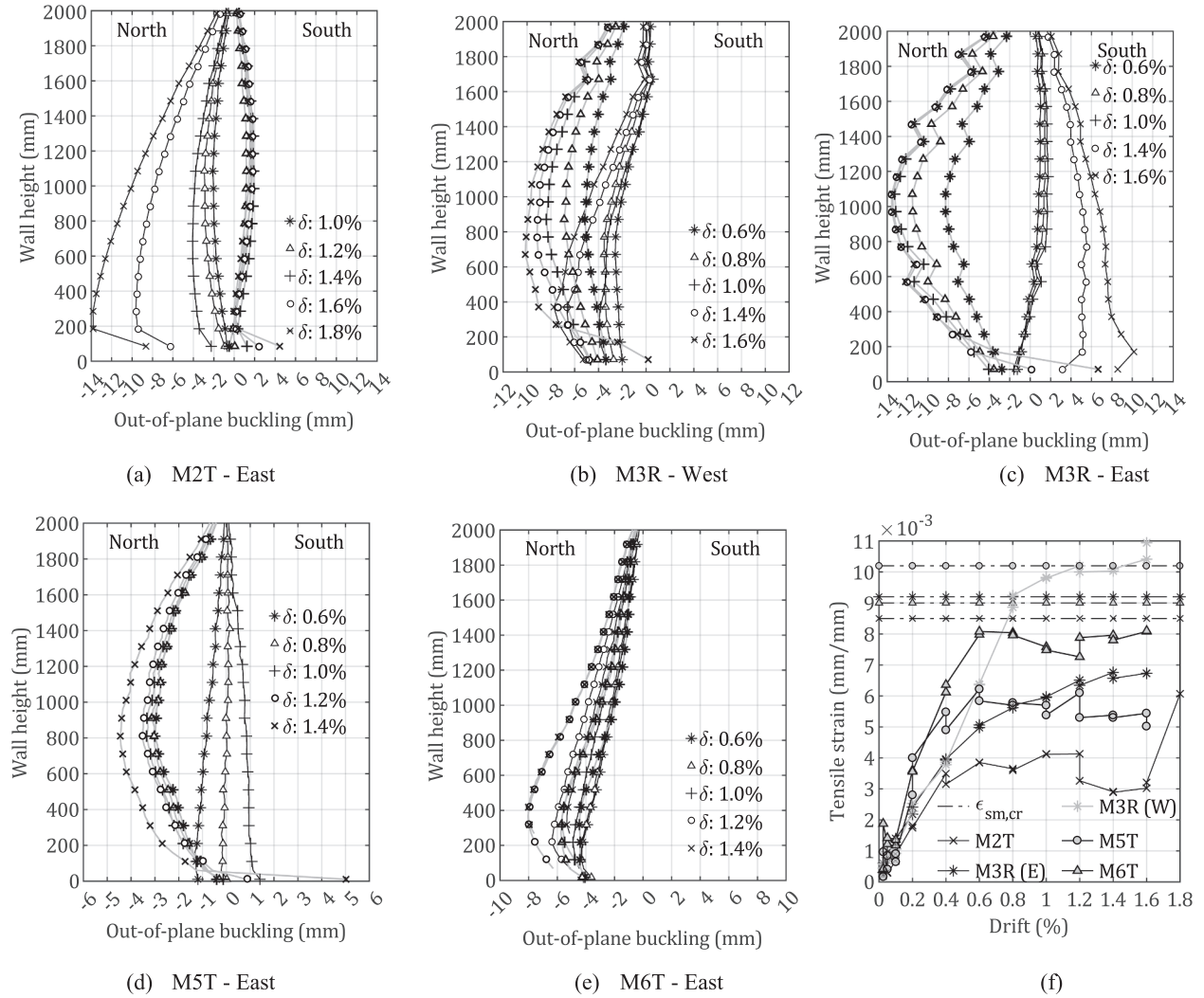


Fig. 15. a) - e) Out-of-plane displacements, and f) maximum tensile strain vs. out-of-plane instability limit.

the height at which the tensile strain is equal to 0.004 for the maximum drift recorded in the strain profiles obtained by photogrammetry, as mentioned above. For a strain of 0.004, higher than ϵ_y , an acceptable value for the estimation of L_{pz} is obtained [32]. Fig. 14a and b show the

strain profiles and the location of L_{pz} with dashed lines for M2T and M3R walls. Table 6 summarizes the results of both methods and includes the damage extent (Fig. 8), i.e., the region over which the concrete spalled [33]. The plasticity distribution was also evaluated with strains

measured directly on the longitudinal reinforcement at the ends of the walls. Strain gauges were installed on those bars at a height of 300 mm (Fig. 4b). Table 6 shows the maximum values for tensile and compressive strains in the external reinforcement, also the drift in which such record was obtained. A good correlation was observed between the height of the plastic zone estimated by curvature and by the strain profile. Damage extent also showed values consistent with the estimated L_{pz} , except for the east end of walls M4I and M5T. However, the tensile strains in the external bars, at a height of 300 mm, reached values higher than $2\epsilon_y$, indicating that the plastic zone was greater than 300 mm, even at the ends of walls M4I and M5T, for which a reduced L_{pz} was estimated. The above is shown in Fig. 14c, which shows the strain record for wall M5T at a height of 300 mm.

4.6. Out-of-plane buckling

Fig. 15 shows the out-of-plane displacement (δ_{oop}) profiles obtained by photogrammetry and validated with the record of the sensors installed to measure this effect along the height of the walls (Fig. 4). All units, except the M4I, exhibited slight signs of out-of-plane buckling. Wall M3R exhibited the largest δ_{oop} , with a maximum value of 13.8 mm, while M5T exhibited the least displacements, with a maximum value of 4.5 mm. In all tests, the maximum values were presented for drift peaks. However, for drifts close to 0%, the displacements were of similar magnitude. In Fig. 15, the profiles of the last drift levels were included to show the progression of out-of-plane buckling. These profiles correspond to the maximum lateral displacement in the plane for each drift level. The black lines indicate the positive direction of loading, and the

the observed behavior. Although greater reinforcement content at the ends is expected greater out-of-plane displacements [24,36], the M3R wall, which had a lower amount of reinforcement than the web end of the M2T, exhibited higher displacements than the latter, being of the same length and with a rectangular cross-section. This was because the M2T wall presented a crack of greater amplitude at the interface (Fig. 12), which caused the tensile strains along the height of the web end to be of a lower magnitude than in the M3R wall. In this wall, the interface crack was the smallest in amplitude, and the tensile strains along its ends were among the highest, which would explain why it was the wall with the largest buckling. In the M5T and M6T walls, whose only difference was the web end reinforcement, and in which the interface crack was similar (Fig. 12), the higher reinforcement content of the M6T (2.97 vs. 1.90%) probably caused the magnitude of the tensile strains in height to be greater than in the M5T, and therefore its δ_{oop} to be slightly higher.

Fig. 15f also included the value calculated with Equation (1) of Parra and Moehle [24] for each wall, which indicates the critical tensile strain that would induce out-of-plane instability, $\epsilon_{sm,cr}$. Comparison of the equation results with the tensile strains correlates well with what was observed in the tests. The M3R wall, with the largest out-of-plane displacements, was close to the limit calculated with the equation (0.0092), while the M2T, with less buckling, was farther away from the limit strain (0.0085). Similarly, the M6T wall was closer to the $\epsilon_{sm,cr}$ (0.0090) than the M5R to its limit (0.0102), and, therefore, the latter had smaller displacements apparently. In the tests of Blandón et al. [3], out-of-plane displacements, normalized by thickness, were reported to be slightly lower than those recorded in this study (i.e., 8.5 vs. 13.8%).

$$\epsilon_{sm,cr} = k\epsilon_{cr} \left(\frac{\pi t_{w,cr}}{K h_w} \right)^2 + 0.005 \quad \epsilon_{cr} = 0.5 \left(1 + \frac{2m}{0.85} - \sqrt{\left(\frac{2m}{0.85} \right)^2 + \frac{4m}{0.85}} \right)$$

$$m = \rho \frac{f_y}{f'_c} \quad (1)$$

gray lines indicate the negative direction. The δ_{oop} values are affected by lateral displacement at the base of the walls due to cracking at the foundation-wall interface.

The δ_{oop} at the web end of the M2T wall did not exceed 4 mm, up to a drift of 1.4% when concrete spalling began. When the reinforcement buckled, for a drift of 1.6%, the maximum δ_{oop} (9.6 mm) was observed at a height of 285 mm. In wall M3R, buckling was recorded at both ends. At the west end, displacements progressively increased to a maximum value of 10.2 mm at the instant of failure in the negative direction (1.6%). At the other end, the δ_{oop} reached a value of 13.3 mm for a 1% drift when web reinforcement fracture and rigid body rotation began. The maximum displacement was 13.8 mm when the wall failed in the positive direction (1.4%). The buckling effect was greater at the east end, which probably, contributed to the failure occurring first in the positive direction. For the west end, the maximum displacements occurred at a height between 670 and 870 mm, and for the other end 1070 mm. At the web end of wall M5T, the displacements reached a maximum value of 4.5 mm for the ultimate drift (1.4%) at a height of 810 mm. In the M6T wall, the maximum δ_{oop} was 8 mm at a height of 370 mm, for a drift of 1.4%, after concrete spalling and buckling of the reinforcement.

The out-of-plane buckling observed in the walls did not cause lateral instability. For the walls with higher δ_{oop} , the buckling may have contributed to the spalling and buckling of the reinforcement. In walls M2T to M5T, the buckling developed over the entire height of the wall, and only in wall M6T, a concentration of displacements was observed in the lower half. Fig. 15f shows the progress of the maximum tensile strain at the ends of the walls from a height of 300 mm. This parameter, which governs the lateral instability of the walls [34–35], helps to understand

5. Conclusions

This article presents the experimental response of five TRCWs subjected to quasi-static cyclic loading. Based on these results, the following conclusions and observations were made:

The test walls reinforced with electro-welded mesh in the web and without effective confinement at the ends, which represent the common practices of the industrialized system in Colombia, showed a moderate displacement capacity for the case of the shorter length walls (1200 and 1500 mm). The ultimate drift for these units was between 1.4 and 1.6%. This ductility is explained by their reduced length and by the amount of longitudinal reinforcement concentrated at the ends that favored the distribution of plasticity along the height of the wall. However, this type of walls could fail for drifts close to 1%. This is the case of the walls M5T and M6T, which exhibited a lower displacement capacity because the premature fracture of the web reinforcement caused a stronger drop in resistance than in the short walls. In the M5T wall, for a drift of 0.8%, the drop in resistance due to this effect was close to 20%, which could have determined its failure. For the same level of drift, the M6T wall had a decrease in resistance of 29%, which caused its failure. In both tests, this event occurred in the direction of the load that subjected the flange to tension. The test database of TRCWs, and the results of this investigation indicate that the displacement capacity for longer walls may be limited compared to the maximum drifts allowed by the design code.

The use of ductile reinforcement in the web of the M2T wall enhanced its hysterical response. Despite having the same amount of reinforcement in the web as the other walls, the properties of the bars,

such as a higher strain-hardening ratio and 3.8 times higher rupture strain than the mesh, led to the fracture of the reinforcement of the web to occur for higher drifts, the impact on the resistance due to the fracture was less critical, and the cycles were more stable. This allowed a greater displacement capacity and a higher energy dissipation compared with units of the same length. Similarly, the wall with special BEs delayed the onset of the fracture of the electro-welded reinforcement and the buckling of the longitudinal reinforcement, in addition to maintaining the integrity of the concrete core. This markedly increased the displacement capacity up to a drift of 2.5%, which is 1.7 times higher, on average, than in the other walls.

The experimental results confirmed that the null or little capacity of hardening by deformation, the limited ductility of the electro-welded mesh, and the low amount of reinforcement in the web as well the use of a single layer of reinforcement, control and limit the displacement capacity of the TRCW, even if they have a significant amount of ductile reinforcement concentrated in the ends. These characteristics of the reinforcement of the web result in the concentration of plasticity in a main crack in the foundation-wall interface that crosses the entire wall, and it becomes considerably widened, increasing the demand for strains in the critical section of the base, which affects the resistance and accelerates the spalling process of the concrete and the buckling of the longitudinal reinforcement, leading to failure.

The high slenderness ratios ($h_w/t_w > 16$) of TRCW and the presence of a single layer of reinforcement can cause lateral instability, especially when they have a considerable amount of longitudinal reinforcement at the ends. However, in the tested walls, the effect of damage concentration at the base was more critical and controlled the failure besides limiting the magnitude of the tensile strains above the base at the ends of the walls, which considerably reduced the out-of-plane displacements.

Author statement

The authors of the article entitled: Behavior under lateral cyclical load of thin reinforced concrete walls of the industrialized system, Roger Ortega, Pedro Torres, Johannio Marulanda, Peter Thomson, and Gilberto Areiza, approve the revised version of the submitted manuscript and guaranteed that the paper contains original research and has not been submitted or published earlier in any journal, and it is not being considered for publication elsewhere.

Ethical procedure

- The research meets all applicable standards with regard to the ethics of experimentation and research integrity, and the following is being certified/declared true.
- As an expert scientist and along with co-authors of concerned field, the paper has been submitted with full responsibility, following due ethical procedure, and there is no duplicate publication, fraud, plagiarism, or concerns about animal or human experimentation.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

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