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EXPERIMENTAL CAMPAIGN OF THIN REINFORCED CONCRETEE SHEAR WALLS FOR LOW-RISE CONSTRUCTIONS

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Abstract

The Chilean reinforced concrete (RC) code (decree DS 60) allows the use of ordinary RC walls for structures up to five stories if the structure is designed with a strength reduction factor equivalent to that of masonry. This clause allows the use of thin walls up to 100 mm thickness, and the use of single layer reinforcement. The local industry is pushing for this type of structures, even though, the seismic performance of these walls have not been proved locally in the last earthquakes. This paper summarizes an experimental campaign conducted to characterize the seismic behavior of thin RC walls with single layer reinforcement for housing applications. The proposed tests are intended to contribute to the understanding of the seismic behavior of thin RC walls with single-layer reinforcement by providing additional tests results. The test program consisted on nine full scale specimens subjected to lateral cyclic displacements and no axial loads. The length and height of the walls were 1600 mm, and the aspect ratio 1.0. The thickness of the reference wall (WSL1) was 100 mm, and it was designed with a single layer of reinforcement. A welded-wire mesh with vertical and horizontal reinforcement ratios of 0.002 was used for the reference wall. The variables analyzed in the test program are the type of steel (welded-wire mesh or deformed bars), the reinforcement ratio, and the wall thickness. Preliminary results are summarized on this paper. Results from these tests may be used to validate the seismic behavior of thin RC walls with single layer reinforcement and for seismic risk assessment of low-rise residential structures and houses with RC walls. Additionally, it is concluded that the ACI 318 equations predicted the strength of the tested walls more accurately than the equations proposed by Carrillo and Alcocer.

Keywords: walls; reinforced concrete; welded-wire, single layer reinforcement; experimental analysis

1. Introduction

The number of residential structures in Chile is approximately 4.26 million. Most of these structures are houses (99.5%) and only 0.5% of these structures are buildings [1]. Houses typically have between one and three stories, and the participation of reinforced concrete (RC) houses in the total number of houses is about 8% (330,000 RC houses) [1]. The participation of RC houses is small compared to that of masonry or wood houses, however, the participation of RC houses have been increasing significantly in recent years. This increase is attributed to the industrialization of the construction process, which has reduced construction costs, and because the society is demanding higher quality construction.

Extensive analytical and experimental research has been conducted in Chile and worldwide regarding the seismic behavior of RC walls for seismic applications. The seismic design of RC walls in Chile is based on ACI 318-08 [2] with some modifications. These provisions are focused on RC walls for mid to high-rise buildings, and special provisions for houses are not included explicitly. RC houses typically have a large density of RC walls, and designing these walls using ACI 318 provisions results in over conservative designs. The Chilean code for reinforced concrete, decree DS 60 [3], allows the use of ordinary RC walls for structures up to five stories if the structure is designed with a reduced strength reduction factor, equivalent to that of masonry. This clause allows the use of thin walls no less than 100 mm, and the use of single layer reinforcement. However, this type of walls has not been incorporated massively in Chile because the seismic performance of thin RC walls with single layer reinforcement has not been proved in real earthquakes.

This paper summarizes an experimental campaign conducted to characterize the seismic behavior of thin RC walls with single layer reinforcement for low-rise construction (i.e. housing applications). The test program of nine full scale specimens is described and some preliminary results are summarized. The walls were subjected to lateral cyclic displacements, without axial load. The length and height of the walls are 1600 mm, and the aspect ratio 1.0. The thickness of the reference wall (W1) was 100 mm and it was designed with a single layer reinforcement using a welded-wire mesh with a vertical and horizontal reinforcement ratio of 0.002. The variables analyzed in the test program were the type of steel (welded-wire mesh or deformed bars), the reinforcement ratio, and the wall thickness.

The load displacement relationships were obtained, the failure modes were identified, and the deformation capacity was determined. The effects of the type of steel, the reinforcement ratio, and the wall thickness on the seismic behavior of the walls were identified. Also, the measured strengths are compared to ACI 318-14 [4] shear strength equations, and other equations for thin walls proposed by Carrillo and Alcocer [5]. Results from these tests may be used to validate the seismic behavior of thin RC walls with single layer reinforcement. Additionally, the test results may be used for seismic risk assessment of houses with thin RC walls.

Experimental research of RC walls for low-rise construction has been reported in the literature. Carrillo and Alcocer [6] subjected six thin RC walls with single layer reinforcement to shaking table test. Quiroz et al. [7] subjected seven full-scale thin RC walls with single layer reinforcement to cyclic loading. Most recently, Bismark et al. [8] tested 12 low-aspect ratio RC walls for low- and medium-rise buildings. However, these walls were detailed with two layers of web reinforcement.

2. Experimental Program

A total of nine thin walls with single layer reinforcement were constructed and tested under cyclic lateral load at the Structural Engineering Laboratory of Pontificia Universidad Católica de Chile. The test matrix is summarized in Table 1. Since the vertical load in low-rise construction is negligible, no axial load was applied to the walls in this experimental program. The length of the walls was 1600 mm, the height 1600 mm ($h_w / l_w = 1.0$), and the thickness 100 mm for six specimens (WSL1 to WSL6) and 80 mm for three specimens (WSL7 to WSL9). The geometry of the walls with 100 mm thickness is shown in Fig. 1. The walls were designed with bottom and top RC beams to connect them to the loading frame. The lateral load was applied at mid height of the top beam (at 1750 mm from the base of the wall) and the resulting shear ratio (M/Vl_w) was 1.09 for all walls.

Table 1 – Test matrix

Wall	Purpose	t_w (mm)	Mesh steel	Mesh detailing	$\rho_t = \rho_l$	ρ
WSL1	Reference wall	100	AT560	ϕ5@100	0.0020	0.008
WSL2	Reduced boundary reinforcement	100	AT560	ϕ5@100	0.0020	0.003
WSL3	Large web reinforcement ratio	100	AT560	ϕ7@150	0.0026	0.008
WSL4	Reduced web reinforcement ratio	100	AT560	ϕ4.2@100	0.0014	0.008
WSL5	Steel type	100	A630	ϕ8@250	0.0020	0.008
WSL6	Steel type and reduced web reinforcement ratio	100	A630	ϕ8@360	0.0014	0.008
WSL7	Wall thickness and large web reinforcement ratio	80	AT560	ϕ5@100	0.0025	0.006
WSL8	Wall thickness and reduced web reinforcement ratio	80	AT560	ϕ4.2@100	0.0017	0.006
WSL9	Wall thickness and steel type	80	A630	ϕ8@250	0.0025	0.006

Wall WSL1 was defined as the reference wall and it was reinforced with a single layer welded-wire mesh. Steel AT560 was used for the mesh with nominal yield strength of 500 MPa. An ACMA C196 welded-wire mesh was specified for this wall and the transverse and longitudinal web steel ratio was 0.002. The transverse steel ratio is equivalent to the minimum specified by ACI 318 [4] for ordinary walls. To simulate construction practice, a short welded-wire mesh was cast with the bottom base, which was spliced with the welded-wired mesh of the wall with a lap splice length of 300 mm. To prevent a flexural failure mode, 6 ϕ16 mm longitudinal bars were detailed at each boundary, which were tied with ϕ6 mm stirrups spaced at 100 mm. The reinforcement detailing of wall WSL1 is shown in Fig. 2. This figure also shows the reinforcement detailing of the bottom and top RC beams. The reinforcement detailing of the top and bottom RC beams of the other eight walls were identical to that of WSL1.

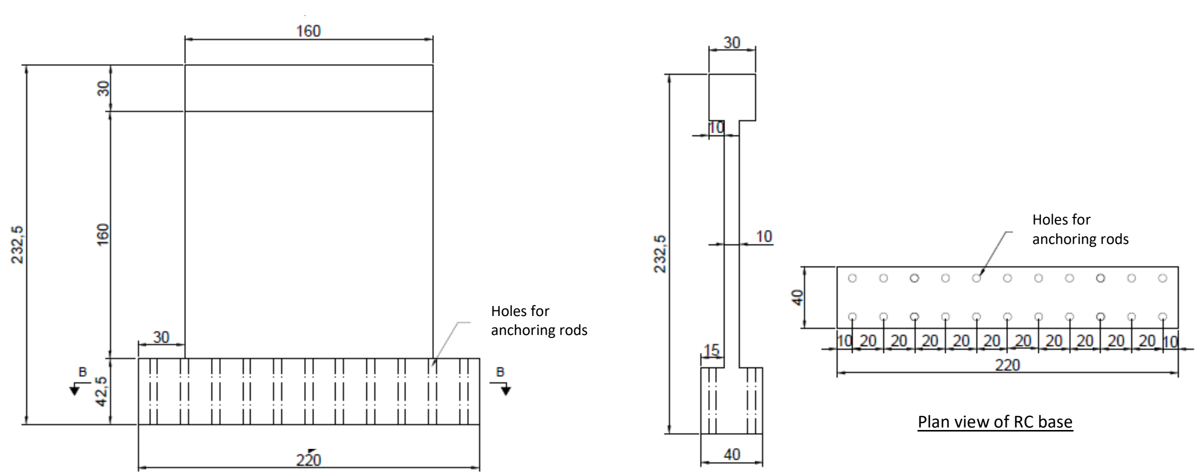


Fig. 1 – Wall dimensions (WSL1 to WLS6)

Wall WSL2 was detailed identical to WSL1 but with 2 ϕ16 mm longitudinal bars at each boundary (ρ = 0.003). Wall WSL3 was detailed with larger web steel ratio (ACMA C257 welded-wire mesh and $\rho_t = \rho_l$ = 0.0026) than WSL1, while WSL4 with a reduced web steel ratio (ACMA C139 welded-wire mesh and $\rho_t = \rho_l$ = 0.0014). Walls WSL5 and WSL6 were equivalent to WSL1 and WSL4, but were detailed with transverse and longitudinal ϕ8 bars as web reinforcement to assess the effect of the type of steel. The resulting bar spacing of 360 mm in wall WSL6 was larger than the minimum of 3*h* = 300 mm required by ACI 318 [4] for walls. The last three walls were constructed with a wall thickness of 80 mm to evaluate the effect of a reduced thickness. Wall WSL7 was detailed with an ACMA C196 welded-wire mesh, and wall WSL8 with an ACMA C139 welded-wire mesh. The resulting transverse and longitudinal web steel ratio of these walls were 0.0025 and 0.0017, respectively. Finally, wall WSL9 was detailed with transverse and longitudinal ϕ8 bars spaced at 250 mm as web reinforcement to assess the effect of the type of steel in an 80 mm thick wall.

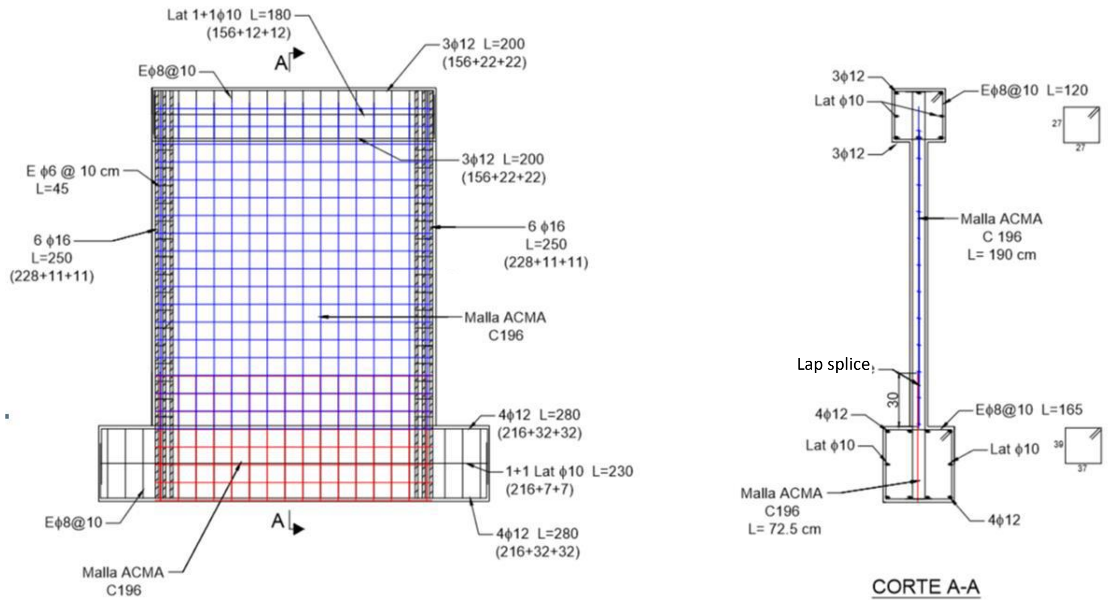


Fig. 2 – Wall detailing, WSL1

2.1. Material Properties

The concrete of the walls was specified with a characteristic compressive strength of 20 MPa and a maximum aggregate size of 10 mm. For the welded-wire mesh, AT560 steel was used with nominal yield strength of 500 MPa. For the deformed bars, A630 steel was used for all bars except for the ϕ6 stirrups at the wall boundaries, where A440 steel was used. The nominal yield strengths of these steels are 420 MPa and 280 MPa, respectively.

The actual properties of the materials were measured in the laboratory. For the concrete, cylindrical samples were tested at the same age as that of the test of the walls. The mean concrete compressive strength was 28.9 MPa and the secant modulus of elasticity 25.4 GPa. The measured modulus of elasticity is equivalent to $4700\sqrt{f_c'} = 25.3$ MPa proposed by ACI 318 [4]. For the reinforcing steel, three bars were tested for each bar diameter. The average yield strength, ultimate strength, modulus of elasticity, and ultimate strain are summarized in Table 2. The table shows the limited ductility of AT560 steel of the welded-wire mesh, where the average ultimate strain is 1.0%, and the ultimate strength is on average only 1.09 times larger than the yield strength.

Table 2 – Average steel properties

Parameter	AT560 Steel (welded-wire mesh)			A630 Steel (deformed bars)		
	ϕ4.2 mm	ϕ5 mm	ϕ7 mm	ϕ8 mm	ϕ12 mm	ϕ16 mm
Yield Strength (MPa)	632	604	601	446	500	438
Ultimate strength (MPa)	680	649	678	685	649	673
Modulus of elasticity (GPa)	182	181	160	196	198	199
Ultimate strain (%)	1.1	0.8	1.2	9.2	9.6	9.4

2.2 Test Setup and Instrumentation

The test setup is shown in Fig. 3, were the walls were bolted at the base with 20 steel rods of 25 mm diameter. The lateral load was applied by a 500 kN hydraulic actuator that was bolted to the top RC beams of the walls. The actuator was pinned at both ends and attached to the top RC beam with four steel rods of 25 mm diameter that were bolted against 400x300x30 mm steel plates at each side of the top beam. The out-of-plane displacement of the walls was restrained with rolling supports that were connected to steel I-beams at each side of the top RC beam. These steel beams are not shown in Fig. 3.

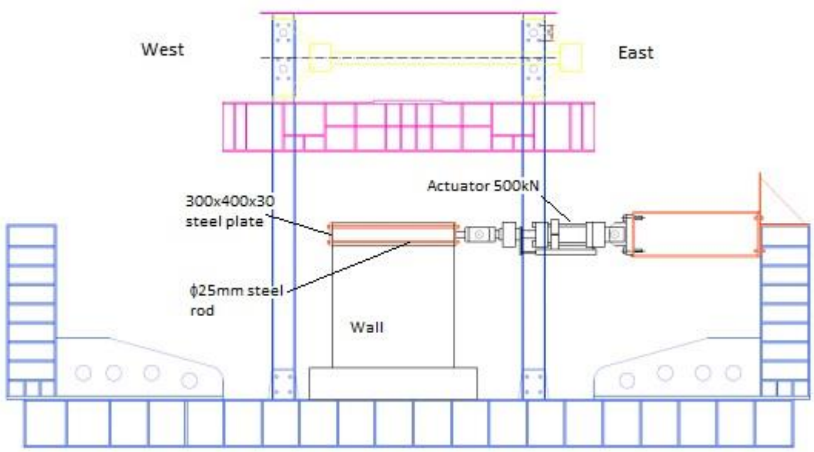
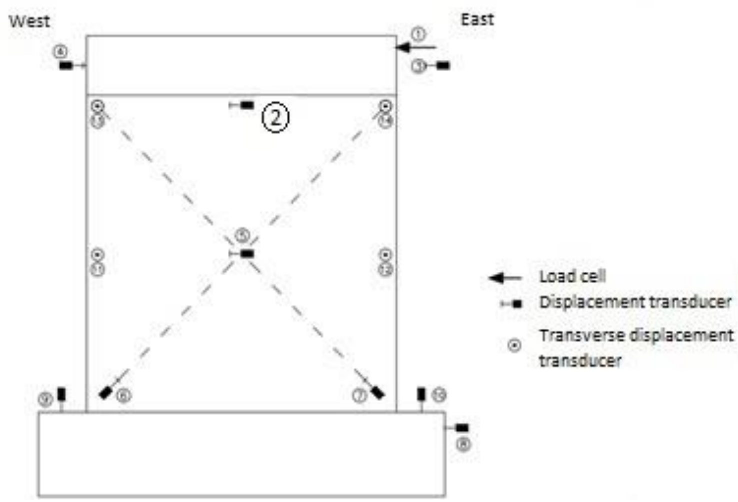


Fig. 3 – Test setup

Each wall was instrumented with one load cell and 14 displacement transducers. The displacement transducers were used to measure horizontal displacements of the wall, the deformation of the two diagonals of the wall, the out-of-plane displacements at mid height and top of the wall, and the displacement and rotations of the RC base. The locations of the displacement transducers are shown in Fig. 4a. The displacement transducers used to measure the lateral and out-of-plane displacements of the wall were attached to a steel frame that was bolted to the RC base of the walls, as shown in Fig. 4b. Therefore, displacement transducers connected to this frame measured displacements relative to the RC base of the walls. Additionally, an SLR camera was used to take pictures of the wall every five seconds. These pictures were used to analyze the displacement field of the walls using image correlation techniques.



a) Locations of instruments

b) Steel frame for the instrumentation

Fig. 4 – Instrumentation

2.3. Load Application and Control

The walls were subjected to lateral cyclic displacements at a constant rate of 10 mm per minute, and no vertical load was applied to the walls. A symmetric quasi-static cyclic horizontal displacement protocol, comprising two cycles at each displacement level was applied. The loading protocol is shown in Fig. 5, where the target drifts at each amplitude level were 0.05, 0.1, 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4, and 1.6%. The displacement transducer used to control the actuator was the #2 in Fig. 4a, which was located at the bottom edge of the top RC beam, at 1600 mm from the base of the wall.

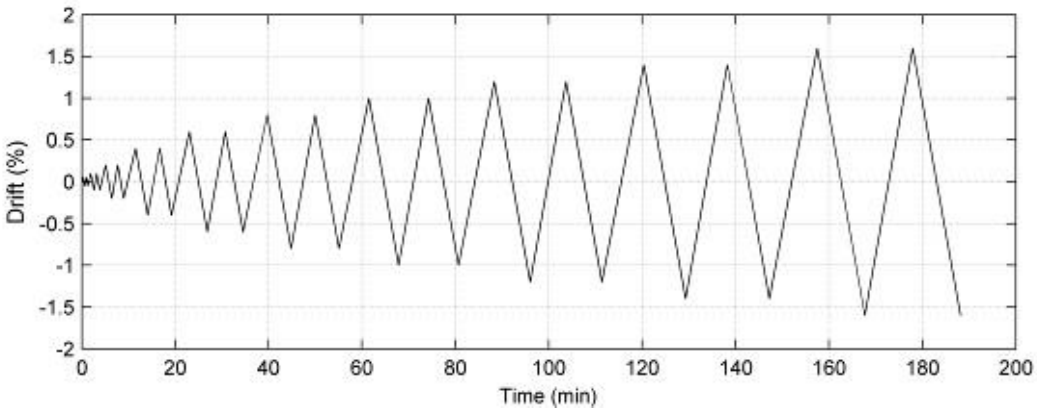


Fig. 5 – Loading protocol

3. Test Results

The test results of the 100-mm thick walls are extensively described by Almeida [9], while the results of the 80-mm thick walls are presented in detail by Pérez [10]. Fig. 6 presents the measured load-drift responses for the nine walls tested. The drift was calculated as the displacement of the top beam relative to the base of the wall (obtained by displacement transducer #2 in Fig. 4a) divided by the distance from the transducer to the base of the wall (1600 mm).

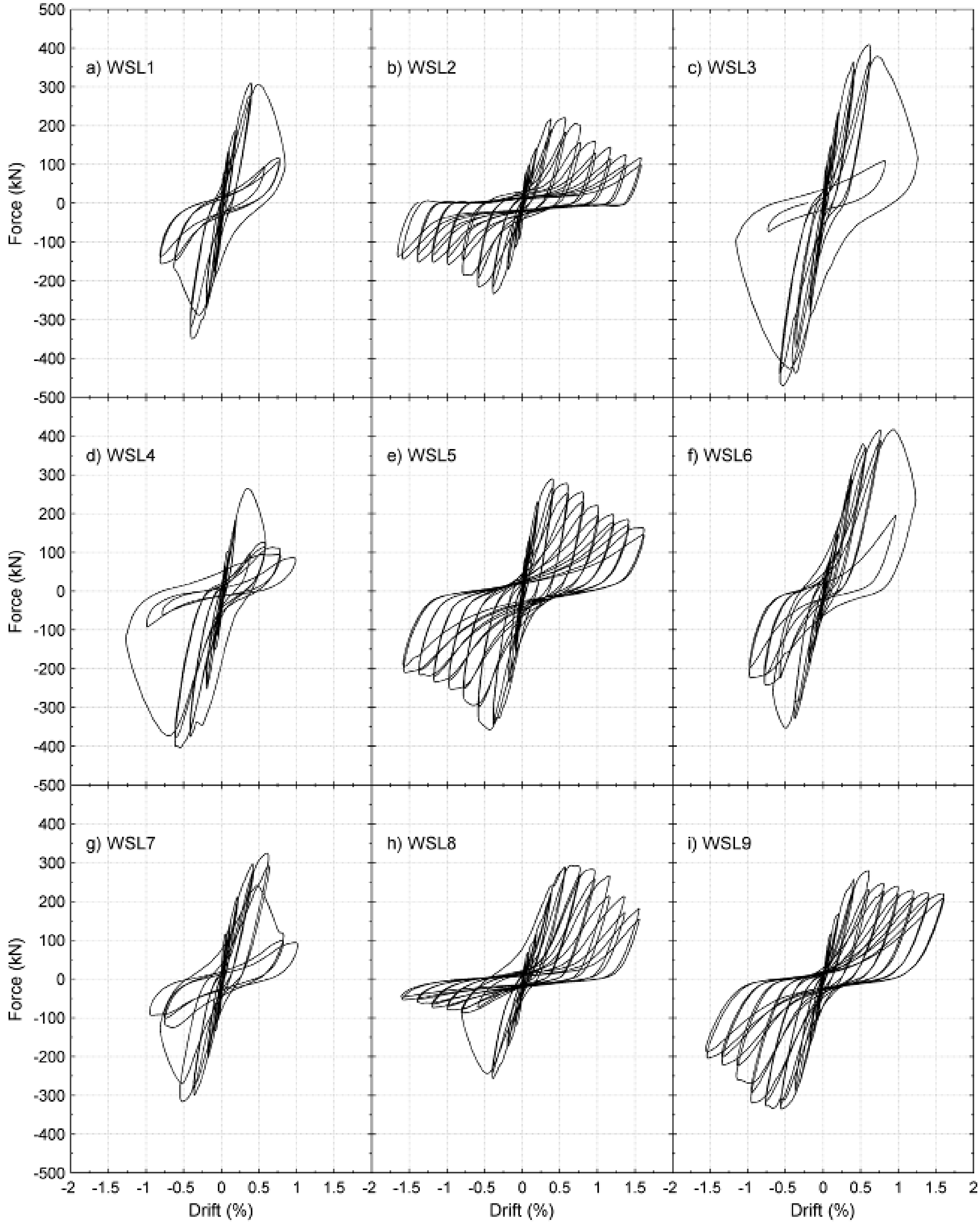


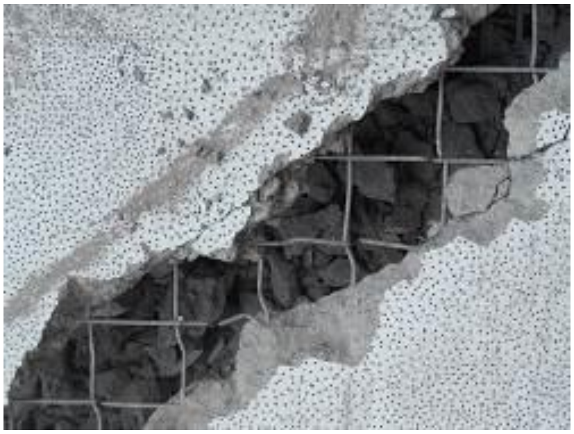
Fig. 6 – Measured load-drift responses

The load-drift responses of walls WSL1, WSL3, WSL4, WSL7, and WSL8, which where detailed with welded-wire mesh as web reinforcement, are similar and have the following characteristics: 1) the initial stiffness does not change up to the force that produces an initial large diagonal crack; 2) after cracking, the secant stiffness slightly decreases and hysteretic response is apparent, showing some pinching of the curves; 3) the maximum load is reached at a drift around 0.5% (see Table 3 for the drift values); 4) after the maximum strength was reached, the horizontal reinforcement fractured and the strength rapidly decreased; 5) with the exception of WSL8, a similar sudden drop of strength was measured in both loading directions; and 6) after the drop of the applied load, a residual strength of 25% to 50% of the maximum load was available at each wall. The fracture of the horizontal web reinforcement is shown in Fig. 7a, after an autopsy conducted to one of these specimens. For walls with different thicknesses or different mesh reinforcement ratios, the described behavior was observed without relevant differences. However, the response of wall WSL8 (Fig. 6h) was different. Fig. 6h shows that the response in the third quadrant is similar to the one previously described, but the response in the first quadrant shows a plateau up to 1.0% drift, and a decrease of the maximum load at larger drifts. This occurred due to dowel action of the boundary reinforcement, as can be seen in Fig. 7b.

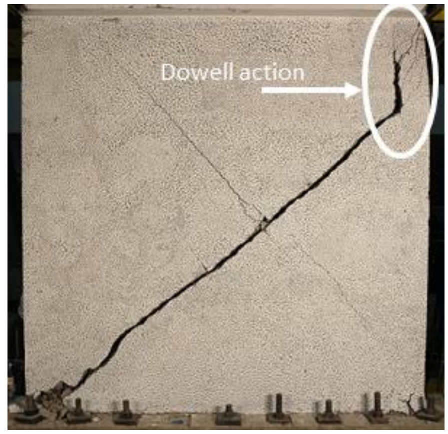
Wall WSL2 was detailed with welded-wire mesh reinforcement, as the described walls, but also with a reduced boundary reinforcement ratio (0.003, as compared to 0.008, Table 1). The initial behavior, up to the maximum strength and a drift of approximately 0.5%, was similar to that of the other walls with welded-wire reinforcement. However, the maximum strength was 30% to 50% smaller than the strength of the other walls with welded-wire reinforcement. For drifts larger than 0.5%, the maximum load in each cyclic steadily decreased to approximately 50% of the maximum strength at a drift of 1.5% (Fig. 6b). Sliding between the wall and the base was observed at drifts larger than 0.5%, which is confirmed by the almost horizontal response at larger drift cycles observed in the corresponding force-drift response curve. Additionally no fracture of the bars of the reinforcement mesh was observed.

Walls WSL5, WSL6, and WSL9 had traditional deformed bars as mesh reinforcement. Walls WSL5 and WSL9 have similar load-drift response (Fig. 6), which initially was similar to the previous walls: 1) the initial stiffness does not change up to the force that produces an initial large diagonal crack; 2) after cracking, the secant stiffness slightly decreases and hysteretic response is apparent, showing some pinching of the curves; 3) the maximum load is reached at a drift around 0.5% (see Table 3 for the drift values); 4) After the maximum strength was reached, the maximum load at each cyclic decreased as the drift deformation increased. The test of WSL5 and WSL9 were terminated after reaching a maximum drift of approximately 1.5%, with a residual strength larger than 60% of the maximum strength. No fracture of the horizontal bars occurred, and no difference in global response was observed due to the different web reinforcement ratios or wall thickness (except for the expected difference in maximum strength). Even though wall WSL6 had the same boundary reinforcement and type of web reinforcement as walls WSL5 and WSL9, the response was different: a sudden loss of strength occurred at a 0.5% drift in the third quadrant, and at 0.9% drift in the first quadrant. The loss of strength in WSL6 was due to fracture of the web reinforcement, which occurred because this wall had smaller web reinforcement ratio (0.14%, Table 1) than the other two walls with deformed bars.

Table 3 summarizes the type of failure, maximum strength, the drift at maximum strength, and the ultimate drift of each wall. The table shows the values measured in both direction of loading. For the walls that had brittle failure, the ultimate drift is equal to the drift at maximum strength. For the walls that had ductile failure, the ultimate drift is defined as the drift at which the measured strength is 20% smaller than the maximum strength.



a) Detail of fractured horizontal bars of the welded-wire mesh



b) Dowel action in wall WSL8 after failure

Fig. 7 – Observed behavior in selected walls

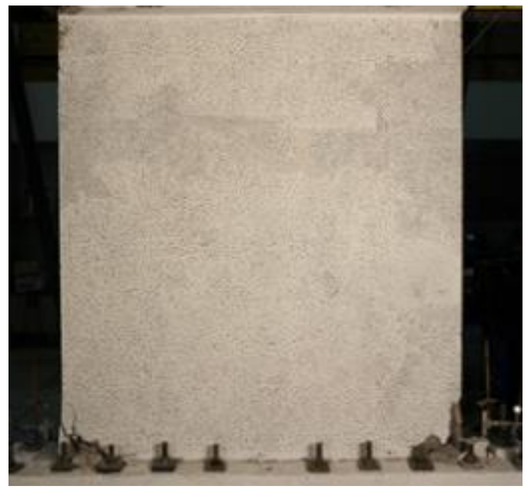
Table 3 – Test results and calculated strengths

Wall	Failure type	Maximum strength (kN)	Drift at maximum strength (%)	Ultimate drift (%)	Shear strength ACI318-14 (2014) (kN)	Shear Strength Carrillo and Alcocer (2012) (kN)
WSL1	Diagonal Tension / Brittle	310 349	0.39 0.39	0.39 0.39	394	285
WSL2	Sliding / Ductile	220 233	0.57 0.38	0.75 0.58	394/313	285
WSL3	Diagonal Tension / Brittle	408 471	0.61 0.53	0.61 0.53	449	323
WSL4	Diagonal Tension / Brittle	265 403	0.34 0.55	0.34 0.55	346	252
WSL5	Diagonal Tension / Ductile	289 357	0.40 0.43	0.80 0.72	350	268
WSL6	Diagonal Tension / Brittle	416 353	0.93 0.50	0.93 0.50	307	234
WSL7	Diagonal Tension / Brittle	316 324	0.61 0.52	0.61 0.52	352	254
WSL8	Diagonal Tension / Brittle	292 256	0.65 0.39	0.65 0.39	304	220
WSL9	Diagonal Tension / Ductile	336 279	0.56 0.60	0.98 1.25	307	236

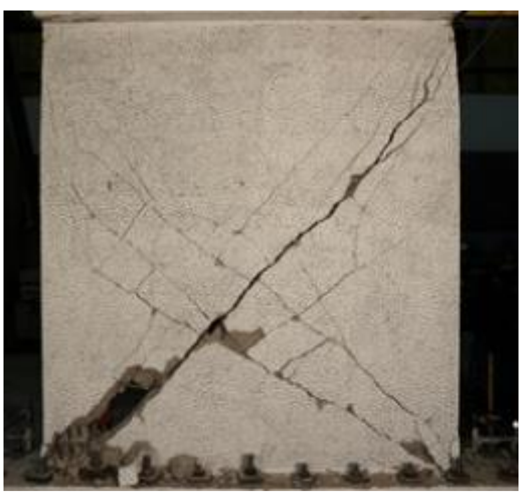
For WSL2 the shear strength (394 kN) and the sliding shear strength (313 kN) are provided.

All walls but WSL2 failed due to diagonal tension, with concentrated diagonal cracks in both directions. As described before, in some cases failure was brittle due to the sudden fracture of the horizontal reinforcement of the web mesh. In the cases with deformed bars, failure was ductile, with a strength plateau and a decrease of maximum force in each increasing drift cycle. No fracture of bars occurred in this case. One wall (WSL2), with a smaller amount of boundary reinforcement than the other walls, failed due to sliding at the base of the wall. The failure modes of selected walls are shown in Fig. 8. Fig. 8a shows the brittle diagonal tension failure mode observed in wall WSL1 where diagonal cracking was concentrated in a single crack in each direction. Fig. 8b shows the sliding shear failure mode observed in wall WSL2, where minimum diagonal crack was observed in the wall. Fig. 8c and d shows the ductile diagonal tension failure mode observed in walls WSL5 and WSL9, were several diagonal cracks were generated in both directions. The cracking pattern through the test of each wall was also obtained using image correlation techniques. Fig. 9 shows the cracking patter for WSL7 at large drift.

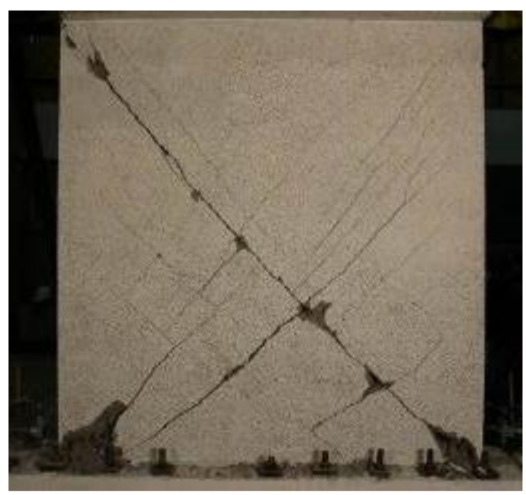
a) Brittle diagonal tension failure, WSL1



b) Sliding shear failure, WSL2



c) Ductile diagonal tension failure, WSL5



d) Ductile diagonal tension failure, WSL9

Fig. 8 – Failure modes of selected walls

All walls, either with brittle or ductile type of failure, reached the maximum strength at drifts levels between approximately 0.35% and 0.55% (minimum values from Table 3). The average drift for the 100-mm thick walls was 0.50%, while for the 80 mm thick walls the average was 0.55%. The walls with welded-wire mesh had an average drift at maximum strength of 0.49%, while for deformed bars the corresponding drift was 0.57%. The ultimate drift of the walls with ductile failure varied between 0.58% and 1.25%, with an average of

0.85%, larger than the ultimate drift of walls with welded-wire mesh, which was equal to the drift at maximum strength.

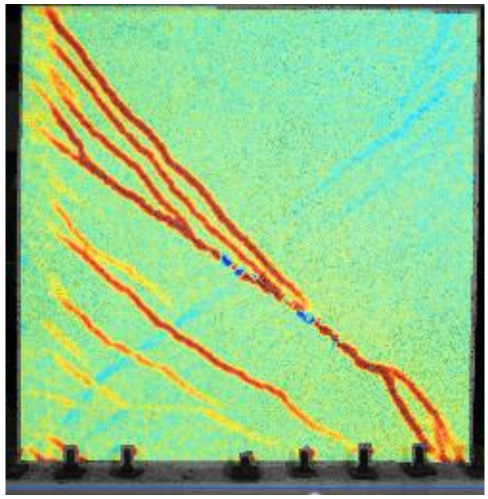


Fig. 9 – Crack pattern of WSL7 obtained from image correlation techniques

The values of the maximum measured strength in the first and third quadrants were always different. The percentage difference between the maximum measured strengths and the average maximum measured strength of each wall is 10% or less, except for wall WSL4, in which that difference is 21%. The wall with smallest strength was SWL2. This wall had boundary reinforcement smaller than all the other walls, which generated a sliding shear failure the base of the wall. In fact ACI 318 [4] equations predict that the sliding shear friction strength is smaller than shear strength for wall WLS2. Finally, the two 100-mm thick walls with web reinforcement ratio equal to 0.0014 (WSL4 and WSL6) had average maximum strength larger than the walls with web ratio of 0.002 (WSL1 and WSL5), irrespective of the type of steel reinforcement which was unexpected.

Equations to calculate shear strength of reinforced concrete walls are provided by ACI 318 [4] in Chapter 18 for earthquake resistance structures. Also, Carrillo and Alcocer [5] recently proposed equations to estimate the shear strength of thin concrete shear walls reinforced with normal and welded-wire reinforcement. Table 3 summarizes the calculated strengths. The equations by ACI318 [4] overestimate the average maximum measured strength of all walls except WLS6. In average, the strengths calculated using ACI318 [4] are 1.04 times the average maximum measured strengths, without considering WSL2, which showed a sliding shear failure mode. The strengths calculated with the equations by Carrillo and Alcocer [5] are all smaller than those calculated with ACI318 [4]. In average, the strengths calculated using Carrillo and Alcocer [5] for the same walls are 0.77 times the average maximum measured strengths. Therefore, it is concluded that the provided equations in ACI 318 [4] predicted the strength of the tested walls more accurately than those proposed by Carrillo and Alcocer [5].

4. Conclusions

This paper summarizes an experimental campaign conducted to characterize the seismic behavior of thin RC walls with single layer reinforcement for low-rise constructions. The variables analyzed in the test program were the type of steel (welded-wire mesh or deformed bars), the reinforcement ratio, and the wall thickness

From the wall tests, the strength and deformation capacity was obtained. Additionally, the failure modes of walls with different configurations were identified. The walls with welded-wire reinforcement mostly exhibited a brittle diagonal tension failure, with concentrated diagonal cracks in both directions, and fracture of the horizontal reinforcement of the web mesh. The walls with deformed bars experienced a ductile diagonal tension failure with distributed diagonal cracking. Finally, the experimental strength was compared with the analytical strength predicted by the expressions of ACI318-14 and by Carillo and Alcocer. In average, the strengths calculated using ACI318-14 are 1.04 times the average maximum measured strengths, and the strengths calculated with the equations by Carrillo and Alcocer are 0.77 times the average maximum measured strengths. Therefore, it is recommended to use the ACI 318-14 equations to estimate the predicted the strength of thin RC walls with single-layer reinforcement.

The tests results validate the seismic application of 100 mm and 80 mm thickness walls with single-layer reinforcement made of deformed bars for low-rise construction (i.e housing applications), because a ductile behavior was achieved. For taller constructions, the axial load in walls is expected to induce lateral instability of thin walls. Welded-wire mesh is inadequate as web reinforcement for seismic applications because the fracture of the mesh implies a brittle behavior of the wall.

5. Acknowledgements

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