



Experimental and numerical evaluation of a U-shaped thin lightly reinforced concrete wall tested under cyclic loading

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Abstract

Reinforced concrete walls provide effective bracing against seismic lateral loading for buildings worldwide. In Latin America, seismic design provisions commonly adhere to the ACI 318 building code, which is predominantly based on United States construction practices. However, in some Latin American countries, the construction methods and geometrical configurations of structural walls significantly differ from those in the U.S.; hence, the available information about the actual behavior of such walls under seismic loads is limited. This study focuses on a thin and lightly reinforced concrete wall (TLRCW) building system, which is characterized by walls thinner than 150 mm and primarily reinforced with a single layer of electrowelded wire steel mesh, with no boundary elements but with additional reinforcing bars at the edges. Past experiments on rectangular and T-shaped walls of the TLRCW building system, which were tested under unidirectional cyclic loading, exhibited limited rotational capacities. This article extends these findings by presenting results from a multidirectional loading test on a U-shaped thin wall and assessing its failure modes, strength and displacement capacity, deformation components, and stiffness degradation. A numerical model based on a nonlinear beam-truss approach was implemented to evaluate the accuracy of the estimates of key performance variables of the wall. The experimental results show limited displacement capacity below 1.15% drift, with a failure mode controlled by concrete crushing at the flange toes. The numerical model was able to capture some of the key global response parameters for all the load directions and at the local level, but with less accuracy.

Keywords U-shaped walls · Thin slender walls · Reinforced concrete walls · Multidirectional test · Beam-truss model

1 Introduction

In regions prone to earthquakes structural walls are widely used as the main lateral load-resisting system for multistory buildings. Historically, design and construction practices for walls in Latin America have been influenced by the ACI 318 building code. However, the adaptation of this code for the countries in this region does not always consider some local

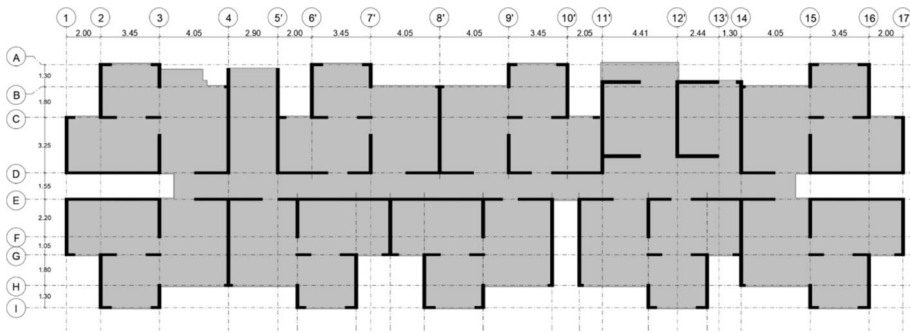


Fig. 1 Typical structural configuration of thin concrete wall buildings

construction practices and material availability, leading to significant deviations in the actual behavior of reinforced concrete (RC) walls under seismic loading from their intended design performances.

The seismic design provisions in ACI 318 come from existing research on RC walls that are considerably thicker and more heavily reinforced than those constructed in some regions of Latin America. This discrepancy raises concerns about the seismic vulnerability of buildings in this region, particularly those employing thin and lightly reinforced concrete wall (TLRCW) systems. These systems typically feature walls that are less than 150 mm thick and utilize a single layer of electrowelded wire steel mesh (WWM) as primary reinforcement. The design usually includes ductile bars at wall edges, but detailed boundary elements to confine the concrete are lacking (Bonett et al. 2024). According to Carrillo et al. (2019) and Miranda-Giraldo et al. (2024), the inherent lack of ductility in WWM significantly compromises the rotational capacity of these walls, resulting in a suboptimal response, as confirmed by Blandon et al. (2018 and 2020) for thin RC walls in Colombia.

According to Bonett et al. (2024), a main feature of the construction method of TLRCW systems is that structural walls are used to configure the architectonical distribution of the apartments; therefore, the geometries of the wall section are often nonrectangular and irregular, as shown in the typical plan of a building in Fig. 1.

The existing research on the seismic performance of thin RC walls in Colombia provides insights into their structural behavior under seismic demand. The studies by Blandon et al. (2018 and 2020) focused on unidirectional reversed cyclic loading tests on rectangular and T-shaped thin walls. These experiments have been instrumental in documenting the mechanical behaviors and failure modes associated with these configurations under seismic loads. Recently, Ortega et al. (2023) extended this scope to include Barbel-type thin walls to consider means to retrofit commonly utilized thin walls in residential buildings. The findings from these studies consistently reveal limited ductility capacities across various wall configurations, highlighting a broader issue of plausible structural vulnerability during seismic events. The consistent observations of limited ductility across these studies indicate an urgent need to revisit design practices and material choices to enhance the earthquake resilience of buildings in this region. Table 1 summarizes some of the key international references found about experimental programs on nonrectangular wall elements that may be related to the case of interest. Key findings are presented next for each study.

Ile and Reynouard (2005) observed good performance, attaining displacement ductility factors of approximately 6 for well-confined walls. They also recognized that while their

Table 1 Specimen features from the literature review

References	name	Cross-section	Scale	t [mm]	⁺ M/VL Xdir/Ydir **	h_u/t	Failure Mode
Ile & Reynouard (2005)	WALL 1	U	1:1	250	3.12/2.6	14.4	Bar buckling and concrete crushing at flange edge
	WALL 2	U	1:1	250	3.12/2.6	14.4	Bar buckling and fracture in tension
	WALL 3	U	1:1	250	3.12/2.6	14.4	Bar fracture after previous buckling, followed by flange shear
Beyer et al. (2008)	TUA	U	1:2	150	2.95 /3.35	17.7	Rupture of the longitudinal reinforcing bars in the west flange
	TUB	U	1:2	100	2.95 /3.35	26.5	Web crushing caused by the reduction of the effective thickness due to spalling of unconfined concrete
Behrouzi, Mock, Lowes, Lehman and Kuchma. (2018)	Wall 7	U	1:3	150	1.2 /3.0	24	Rupture along the center of the web, buckling of the vertical bars at the boundary elements and rupture of bars in the west flange boundary elements
	Wall 8	U	1:3	150	1.2 /3.0	24	Bars ruptured along the East flange, and the specimen could no longer carry the full axial tension
Constantin & Beyer (2016)	TUC	U	1:2	100	2.58 /2.81	17.7	Out-of-plane buckling and compression failure of the west flange
	TUD	U	1:2	100	2.58 /2.81	17.7	Explosive compression failure of the concrete in the east flange
Almeida et al. (2017)	TW4	T	2:3 -1:1	80	22.7/3.7	25	A sudden failure involved abrupt concrete crushing and buckling of the rebars in the boundary element of the web edge
	TW5	T	2:3 -1:1	120	16.7 /2.72	17	A failure involved abrupt concrete crushing and buckling of the rebars in the boundary element of the web edge
Bruggen, French & Sritharan (2017)	NTW1*	T	1:2	152	4.33 /3.46	19.9	Crushing of the flange tips, causing a substantial loss of capacity
	NTW2*	T	1:2	152	4.33 /3.46	19.9	Loading with the flange in tension caused failure of the web tip
Hoult, Appelle, Almeida & Beyer (2020)	TUE	U	1:1	100	3.58 /4.05	42.5	Local out-of-plane buckling in the east flange
Hoult, Doneux & Almeida (2023b)	UW1	U	1:2	100	6.4	22.5	Concrete core crushing and buckling in the boundary end. Out-of-plane instability of the bottom region of the flange
	UW2	U	1:2	100	Torsional test	22.5	boundary end crushing and out-of-plane bending of longitudinal rebars

t is the thickness of the wall

h_u is the unrestrained height of the wall

⁺M is the overturning moment, V is the lateral load, and L is the test unit length in the direction of the load

* walls with intermediate slabs

** X is the direction aligned with the flanges

numerical models correlated well with experimental data, steel buckling was not effectively captured, indicating limitations when faced with bidirectional loading conditions. Beyer et al. (2008) showed the impact of wall thickness on seismic response, especially when subjected to diagonal loading, a condition under which U-shaped walls demonstrated reduced displacement capacities. Behrouzi and colleagues (2018, 2020) provided evidence that bidirectional loading can hinder the ductility capacity of U-shaped walls, with early damage manifesting as concrete spalling and crushing and failure triggered by steel fracture postbuckling. Constantin and Beyer (2016) highlighted the inadequacy of the plane-section assumption for diagonal loading scenarios and the importance of diagonal directions for the design of shear. Bruggen et al. (2017) demonstrated that the height of T-shaped wall specimens and the confinement of longitudinal reinforcement critically influence the in-plane load capacity and displacement. Finally, Hoult et al. (2018) investigated thin RC U-shaped walls with single reinforcement layers, underscoring the necessity for dual reinforcement layers and boundary confinement to prevent out-of-plane buckling and improve flexural compression capacity. Hoult et al. (2023a, b) reported concrete crushing and out-of-plane instability at the flange toes during cyclic loading for lightly confined U-shaped walls at drifts of approximately 2.0%.

These studies demonstrate that U-shaped walls behave differently from traditional rectangular walls, particularly concerning reinforcement concentration and multidirectional load resistance. The unique local construction methods, such as the use of WWM as the primary reinforcement with lightly and concentrated ductile reinforcement at the edges, require further exploration, especially under protocols that induce simultaneous displacements along both principal axes. This article presents the main findings of an experimental study on the characteristics of an RC thin U-shaped wall reinforced with single layers of nonductile WWM subjected to pseudostatic multidirectional cyclic loading. A nonlinear beam-truss numerical model is calibrated with the experimental results to reproduce the main global and local response characteristics. The results confirm the need for addressing some designs, as the displacement capacity was low, with brittle failure modes such as concrete crushing. The numerical results also show highly inconsistent results from models that assume that the plane sections remain planar.

2 Experimental program

A half-scale 1.3 m × 1.3 m U-shaped wall was tested in the structural mechanics laboratory at EIA University. It consisted of a two-story unit with a total height of 2.88 m (Fig. 2a) and typical reinforcement detailing and materials used in TLRCW buildings in Colombia (Bonett et al. 2024). The interstory clear height was 1.2 m for the first story. An intermediate slab was included to capture the lateral bracing effect on the walls and anchor the wall wire mesh, which is a customary detail for the buildings that use this structural typology. A top beam was also included at the top of the wall, which served as a load transfer element to the rest of the wall. To the best of the authors' knowledge, there is no data in the literature on walls similar to the prototype studied in this paper that combine a reduced thickness and WWM as reinforcement in a single layer.

The scaled thickness of the wall was 60 mm, and the reinforcement comprised a single layer of electrowelded wire mesh, 4 mm in diameter, with a center-to-center spacing of 75 mm in the horizontal and vertical directions. The provided longitudinal reinforcement ratio was 0.26%, which is close to the minimum required by the current Colombian

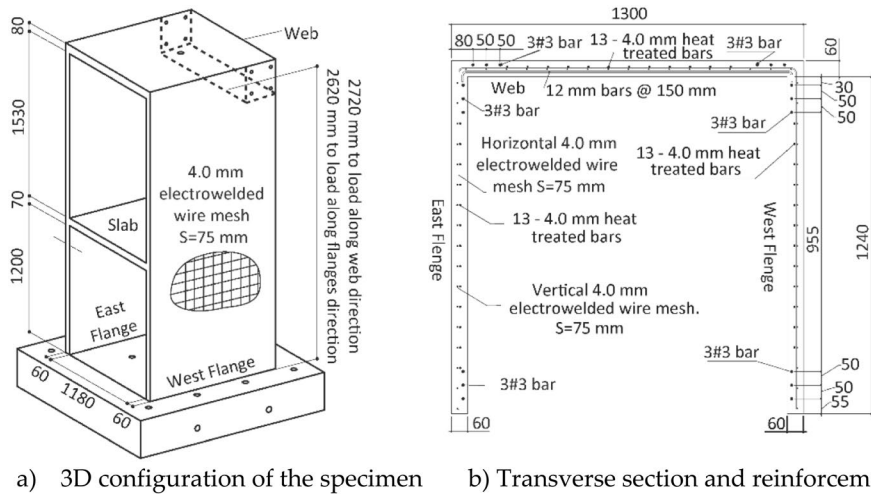


Fig. 2 3D configuration and transverse section of U-shaped wall tested

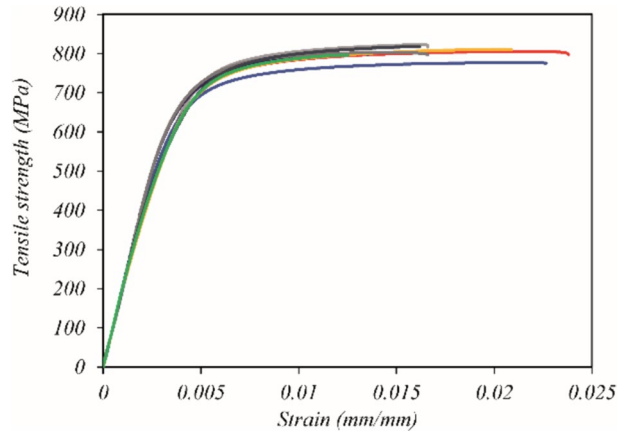
building code, NSR-10 (AIS, 2010). The flanges and the web had additional continuous 9.5 mm (#3) ductile reinforcement at the edges, from the foundation to the top slab (Fig. 2b), resulting in a 2.1% reinforcement ratio at the edges over an area of approximately $3.4 t_w$ (the total length of the section where the additional reinforcement was placed at the edges) $\times t_w$, where t_w is the thickness of the wall. The concrete had no confinement reinforcement following the typical details found in actual TLRCW buildings.

The electrowelded mesh had a 300 mm lap splice from the wall-foundation interface to connect the 4 mm starting dowels embedded in the foundations. The web had an additional 12 mm (#4) of transverse reinforcement bars spaced 150 mm along the height to prevent shear failure of the wall when loaded in the direction parallel to the web, as this is not an expected failure mode based on the design procedure of the current code. According to the calculations for designing the wall, no additional transverse reinforcement was necessary for the flanges. The resulting transverse reinforcement ratio in the web was 1.6%, while it was 0.28% in the flange.

2.1 Mechanical properties of the materials

The concrete used a scaled aggregate of 9.5 mm (3/8") maximum size to produce a concrete with a specified compression strength (f'_c) of 25 MPa. The 9.5 mm thick (#3) ductile reinforcement bars had an average yield strength of 456 MPa and an ultimate strength of 645 MPa. The measured average rupture strain was 14.6%. Figure 3 shows the 4 mm WWM stress-strain experimental curves. Their properties are significantly different from those of typical ductile reinforcement steel, with a maximum average tensile strength of 803 MPa and a limited rupture strain of 1.7%, which shows the fragile nature of this type of reinforcement. Furthermore, Carrillo et al. (2019) argued that the flat postyield nature of the WWM response hinders the propagation of plasticity through the cracking of RC members.

Fig. 3 Stress–strain envelope for the 4.0 mm electrowelded mesh



In actual buildings, the dowels used to lap splice the electrowelded mesh generally have ductile or moderately ductile characteristics; therefore, it was necessary to apply heat treatment to the fragile 4 mm steel wires that were used as scaled dowels to represent the actual reinforcement used at construction sites. After several iterations of heat treatment, the average yield strength obtained was 590 MPa, and the maximum average strength was 636 MPa. The average rupture strain was 8.9%. The entire wall was cast in a single pour in a horizontal position. After reaching the specified concrete strength, the specimen was transported and installed in the laboratory.

2.2 Test setup and instrumentation

A constant vertical load was applied to the top slab using a hollow hydraulic jack and a vertical rod anchored to the foundation (see Fig. 4a). A 500 kN actuator attached to a braced steel frame applied a lateral load parallel to the web (EW direction) at the location of the transfer beam at 2.72 m from the wall–foundation interface. Two 700 kN actuators were attached between the reaction wall and the transfer beam, and lateral force was applied parallel to the flanges (in the NS direction) 2.62 m from the foundation to avoid interference between the attached bolts. This distribution of the actuators allowed the bidirectional load to be applied and the torsional response of the wall to be restrained.

A total of 59 sensors were used to measure displacements and deformations at different key locations of the wall, in addition to 7 sensors used to control the displacement and forces applied by the hydraulic actuators. The first floor of the wall was monitored using seven vertically distributed potentiometers at each of the four edges of the wall to record the axial deformation distribution there (see Fig. 4b). Six string potentiometers were connected from a reference pole to the free edges of the flanges to measure out-of-plane deformations during NS load cycles, with three sensors each on the west and east flanges. Sliding potentiometers were located at both the flanges and the web at the wall–foundation interface. A sensor array of two diagonal and two vertical potentiometers was also installed on the first floor at both the flanges and the web to measure the shear deformations. A single vertical potentiometer along the four edges was located on the second floor to capture axial deformations at that height. In addition to the sensors, the first story of the east

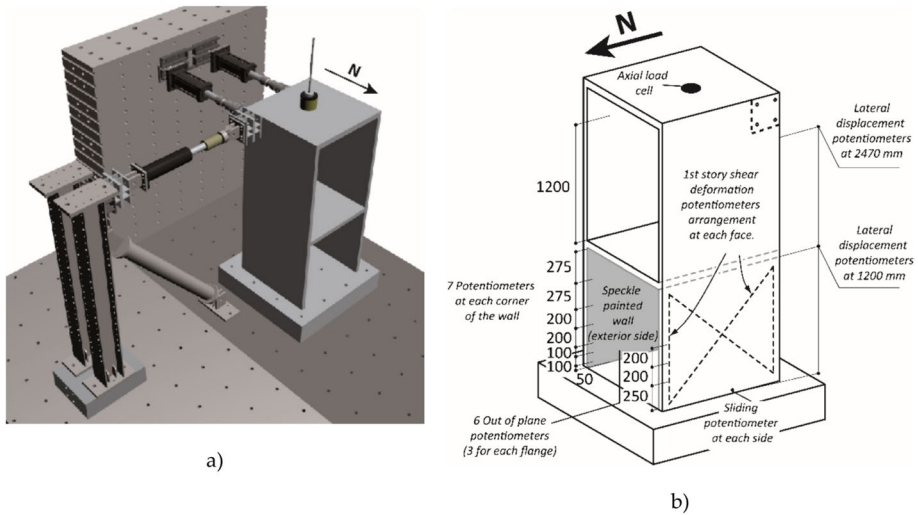


Fig. 4 Test set up for the bidirectional test (left) and instrumentation (right)

face of the specimen was prepared with a speckle pattern to capture the cracking evolution through digital image correlation (DIC).

2.3 Loading protocol

The loading protocol included a combination of unidirectional and bidirectional cycles aimed at evaluating the displacement capacity of the wall at various loading angles. The wall was continuously subjected to a vertical load of 250 kN. The maximum displacement of the cycles (and corresponding drifts) were 1.0 mm (0.04%), 2.8 mm (0.12%), 5.6 mm (0.23%), 8.4 mm (0.35%), 11.3 mm (0.47%), 17.0 mm (0.70%), 22.6 mm (0.93%), 28.5 mm (1.18%), 34 mm (1.40%) (see Fig. 5). For each predefined displacement limit, the protocol

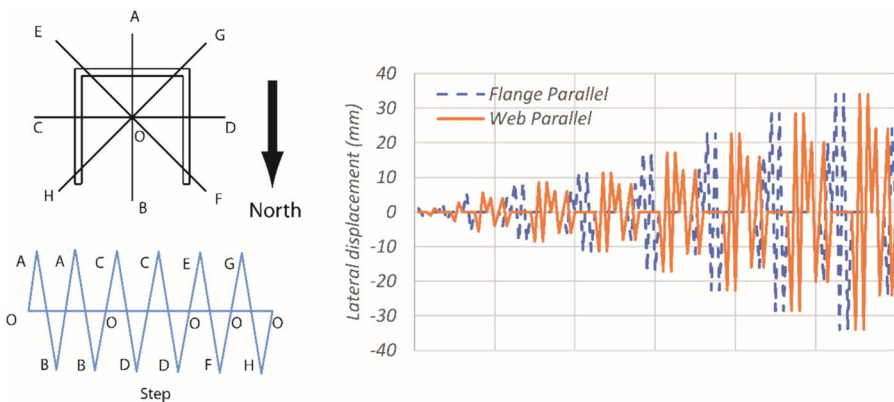


Fig. 5 Loading protocol. Directions and multidirectional pattern (left) and applied displacement history (right)

began with two cycles applied parallel to the flanges (AB, corresponding to the NS direction), followed by another two cycles applied along the web (CD, corresponding to the EW direction). After reaching a roof drift of 0.23%, the regimen progressed to include one cycle along each diagonal (EF and GH). The hypotenuse of the horizontal displacements in the AB and CD directions determined the diagonal displacement magnitude. The cycles intentionally induced tensile strains on the flange edges, aiming to induce elongation of the steel at these locations before inducing compressive deformations during the cycle reversal. This is a critical case, as this condition may trigger lateral instability of the slender walls (Rosso et al. 2016).

3 Measured response

3.1 Cracking pattern

Figure 6, which presents the vertical strain field (ϵ_{yy}) on the bottom (first story) of the east flange across different loading directions at the peak of the cycles just before the initial evidence of concrete cover spalling was observed. Due to space constraints in the lab, it was not possible to install additional cameras to do the same for the west flange and for the web. The sequence was processed using the DIC technique (Suton et al., 2009). This technique maps surface strain by comparing successive digital photographs of a specimen under stress to effectively visualize the emergence and spread of cracks. The visualization traces of strain development through color gradations, where cold colors represent compressive strains and warm colors indicate tensile strains. These chromatic cues provide a clear delineation of the crack patterns across the first story panel. Notably, zones exhibiting high compressive strains appear in dark blue and magenta, while large cracking demand appears red, particularly absent in direction C (toward the east flange) due to the compression in the panel.

For all the sides of the wall, there was a gradual increase in cracking spreading as the drift increased, resulting in a well-distributed crack pattern that spread from the wall-foundation interface to the upper section of the second floor, as shown in the DIC for the first story panel in Fig. 6 and in the cracks distribution images at the end of the test in Fig. 7. This behavior confirms the benefits of additional ductile reinforcement at the wall edges, as first observed by Blandon et al. (2018) in rectangular walls with similar reinforcement. One characteristic feature of the crack distribution around the three sides of the wall was a base crack with a width slightly larger than the web cracks in the wall (see Fig. 6 for directions A, B, D, E and G).

For directions A and B, the flanges presented an approximately 45° slope cracking path, extending downwards from the web side towards the toe of the flange side, for the loading cycles imposing tension on the web (Fig. 6 direction B). For the loading cycles imposing compression on the web (Fig. 6 direction A), the angle of the cracks was between 30 and 40°, opening initially at the free side of the flange and extending toward the toe of the flange-web intersection. This difference in the crack angle was associated with the larger neutral axis during the cycles that induced compression on the flange toe and tension on the web during the parallel flange cycles (NS direction). At the wall-foundation interface, the sensors captured a maximum aperture of approximately 10 mm at the flange toe and 1.6 mm at the web-flange intersection toe for the flange parallel cycles (NS direction).

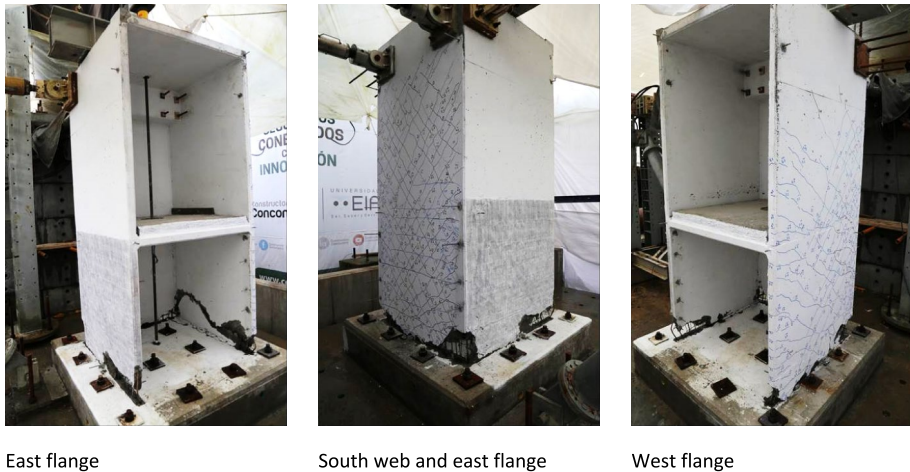


Fig. 7 Wall failure at end of test

were distributed along the height, with the largest cracks occurring at three different locations, namely, the wall foundation interface, 200 mm from the interface and approximately 600 mm from the interface.

In directions B and H, the cracking patterns are notably steep and inclined, with significant compressive strains observed at the base, particularly at the edge of the flange. These compressive patches extend vertically for approximately five times the thickness of the wall, with a higher concentration in the lower half. Direction F presents a cracking pattern akin to B and H, albeit with somewhat relieved compressive strain, as this direction favors tensile strain development on the observed face. Direction C stands out as an exception, showing no visible cracking due to the face being under compression.

Direction E reveals a concentrated compressive patch spanning approximately $3t_w$ (horizontally) by $3t_w$ (vertically) at the bottom left corner, where the flange and web converge. Notably, the compressive strain demand at the web-edge interface is greater for direction E than for direction A, as part of the west flange steel is also excited in tension for the former.

Vertical strain profiles are worth studying to understand the extent of the compressive strain demand along the flange length. Compressive strains are of interest because concrete crushing is likely and may lead to a sudden loss of lateral load resistance in RC thin walls (Arteta and Moehle 2023). This is because special detailing for boundary regions is lacking due to thickness constraints; hence, limit strains for plain concrete may command the response. Figure 6 a–d show the vertical strain profile along the flange length at the same instants described for Fig. 6 (top). Only the directions where compressive strains are induced at the bottom of the first story of the east face are considered. The average strains are estimated from the DIC data and are measured over a gage length of 300 mm ($5t_w$) from the wall/foundation interface. Observations from these figures are as follows:

- Dir-B shows a strain profile for the loading direction that engages in tension in both the web and the web/flange interface ductile additional reinforcement. The relatively shallow neutral axis depth at 23% of the flange length is contrasted by limited tensile strains on the opposite side due to the nonlinearity of the strain profile. Note the divergence of this strain profile with the generally accepted Euler-Bernulli assumption of

linear deformation profile used for structural design. The strain profile in Dir-B shows an approximately constant vertical strain around the central section of the web and a rapid increase of the tensile strains toward the web-flange connection, which are characteristic of the shear lag phenomenon for non-rectangular elements.

- Dir-C shows a strain profile under pure compression, as anticipated.
- Dir-E and Dir-H exhibit a deep compression block within the range of 36–47% of l_w . This suggests that diagonal loading engages a more substantial portion of the longitudinal steel compared to the nondiagonal (NS) direction, leading to greater compressive strain demands.

3.2 Failure mode

The damage that caused failure initially appeared as concrete spalling at the toe of both flanges during the load step that induced compression at this location for a 0.7% lateral drift. The damage increased, reaching concrete compressive crushing and exposure of the longitudinal bar during the second 0.93% drift cycle. During this cycle, there was evidence of fracture of the 4 mm steel connecting the foundation to the wall when the flanges were subjected to tension and concrete spalling at the toe of the flange-web intersections. The 9.5 mm (#3) longitudinal bars at the edges buckled during the following cycles, along the crushed concrete regions of approximately 160 mm at the toe of both flanges; however, they did not fracture. Failure, defined as a force reduction below 80% of the maximum lateral strength, occurred during the second cycle of the 1.18% lateral drift. Further concrete crushing and reinforcement exposure occurred at the flange-web bottom corner edge during this drift. The buckling of the exposed longitudinal reinforcement at the crushed flange toe was evident and increased during each lateral drift increment. The total failure of the specimen occurred at a lateral drift close to 1.4% due to a sudden rupture of the west flange during a diagonal cycle in the northwest direction – from E to F- (see Fig. 7). This cycle induced combined out-of-plane shear–compression failure in this flange.

3.3 Hysteretic response

The hysteresis cycles are presented for each of the load directions in Fig. 8, where the drift was calculated as the ratio of the lateral displacement measured at the location of the sensors and the height of these sensors to the wall-foundation interface. The cyclic response of the walls showed stable hysteretic cycles with negligible strength degradation up to the first 1.18% drift for the cycles in the NS direction (parallel to the flanges). During the second cycle of the 1.18% drift, for the cycles parallel to the flanges (NS direction) and inducing compression in the flanges, severe strength degradation close to 25% of the peak load was associated with extensive crushing of the concrete at the flange toe, and buckling of the longitudinal reinforcement at the same location was observed (see Fig. 8a). The cycles also showed slight pinching, which was more evident in the direction of loading parallel to the web (EW direction) (see Fig. 8b). The maximum lateral load capacity occurred during the flange parallel cycles (NS direction) for a flange compression step of the cycle, which is consistent with the reinforcement distribution and the amount of steel in tension.

The forces in the diagonal direction were obtained as the squared root of the sum of the squares of the EW and NS force components and the direction of the resultant force was verified based on the direction of the force applied by the actuators. The maximum strengths along the NW (toward the F) and NE (toward the H) diagonal are similar, which

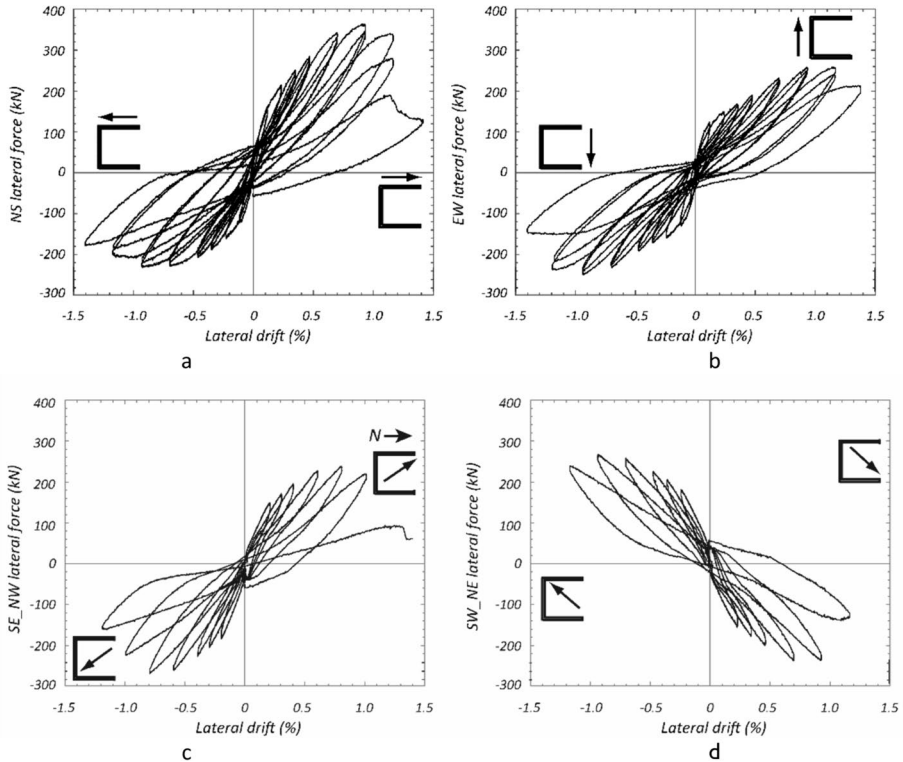


Fig. 8 Hysteresis cycles for the tested wall

is also the case for the SW (toward the G) and SE (toward the E) diagonal. The strength degradation is evident after the concrete crushed at the flange toes during the previous flange parallel 1.18% lateral drift cycles (NS direction). The failure occurred during the first 1.4% lateral drift cycle in the NW direction, when the west flange was in compression, after reaching this drift in the EW direction. There is a slight strength degradation just after the 0.93% lateral drift, which is associated with the initial damage of the concrete and the rupture of some of the 4 mm steel bars. This degradation gradually continued for all loading directions, as concrete crushing was evident at the four corners of the wall until complete fracture of the west flange occurred during the 1.4% drift cycle in the NW direction.

3.4 Strain and curvatures analysis

The strains and curvatures were calculated to analyze the deformation distribution in the wall using measurements from potentiometers installed at the four corners of the wall. The estimation was carried out for lateral drifts, which provided reliable data for the calculations. Figures 9 and 10 show the curvatures obtained for different drift levels and load directions, where the red bars correspond to discrete measurements at various points along the height of the wall at the locations of the potentiometers shown in Fig. 4b. The blue bar was obtained by measuring the average strain over the entire height of the first floor of the specimen using a potentiometer with one edge anchored

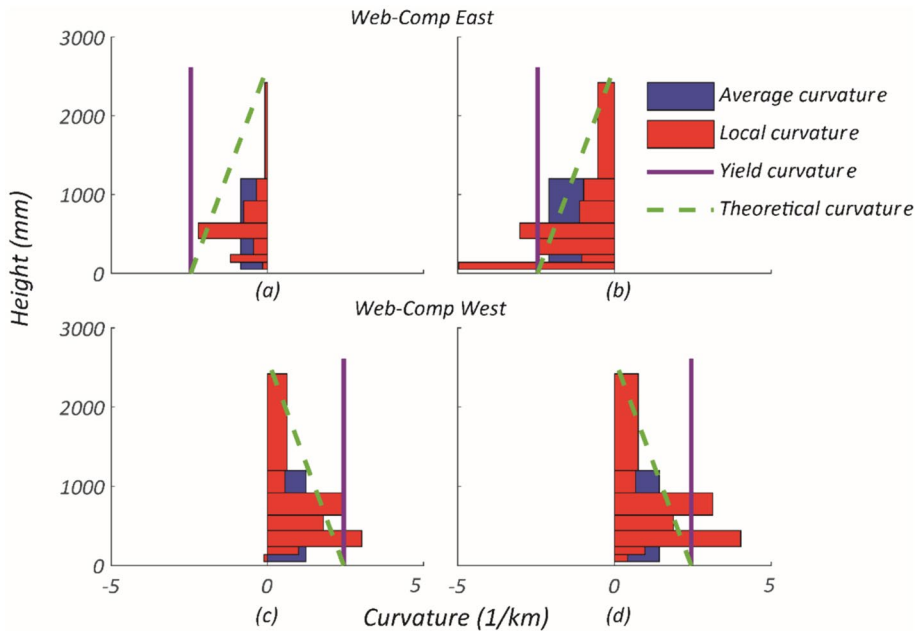


Fig. 9 Curvature distribution for web parallel cycles (EW direction). **a** web curvature for 0.35% drift and **b** 0.47% drift during the cycle inducing compression in the east flange. **c** web curvature for 0.35% drift and **d** 0.47% drift during the cycle inducing compression in the west flange

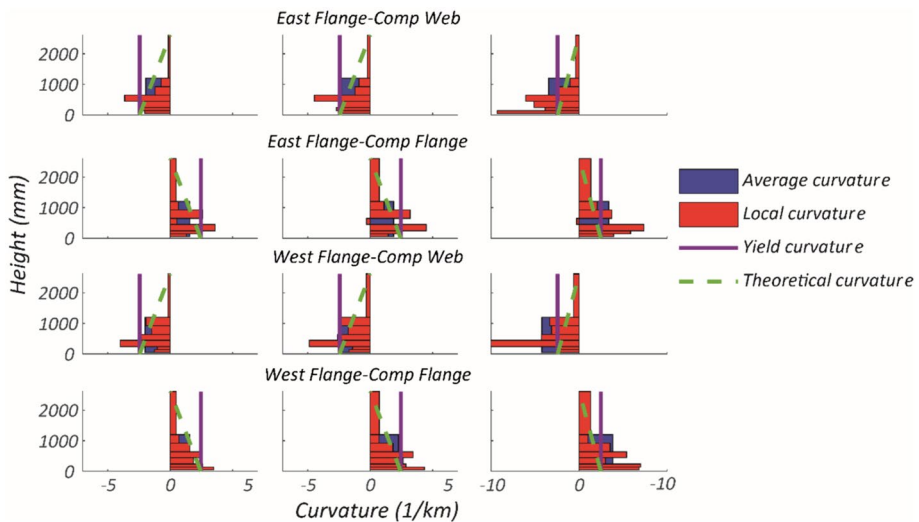


Fig. 10 Curvature distribution for flange parallel cycles (NS direction) at the flanges during maximum web compression and maximum flange compression, for 0.35% (left column), 0.47% (center column) and 0.93% (right column) lateral drifts

just above the wall-foundation interface and the other edge anchored just below the intermediate slab. The curvatures were calculated as the ratio between the sum of the unit deformations at the ends of each element (web and flanges) and the horizontal separation between the potentiometers.

Figure 9 shows the maximum curvature for the web parallel cycle; for lateral drifts of 0.35% and 0.47%, as for larger drifts, some sensors did not record reliable data, and this parameter could not be computed. Even if the curvature distribution is not symmetric in the two directions of loading, the curvature distribution shows how inelasticity distributes along the first floor, up to a height of 600 mm for the east flange compression step (lateral load toward the east) and up to 875 mm for the west flange compression step (lateral load toward the west) for the 0.47% drift.

To evaluate the level of deformation that the wall achieved, the elastic curvature of the wall was calculated using Eq. 1 proposed by Sullivan et al. (2012):

$$\varphi_y = Ki \frac{\epsilon_y}{L_f} \quad (1)$$

where Ki is equal to 1.4 and 1.8 for the web and flange under compression, respectively, for flange parallel cycles (NS direction) and Ki is equal to 1.4 for web parallel cycles. ϵ_y is the yield strain of the steel (0.00228), and L_f is the length of the flange.

The distributions of the curvatures obtained experimentally are shown in Figs. 9 and 10. The yield curvature and a theoretical linear curvature distribution profile are also included for comparison. The average strains do not reach the theoretical yield curvature (solid line); however, at the local level, there are several locations along the first floor that exceed this yield curvature. It is also clear that the actual local curvature distribution at the first floor diverges significantly from the theoretical curvature distribution (dashed line), in particular at the lowest section of the wall. This may be partly associated to the lap splice in the lowest 300 mm sections, which nearly doubles the reinforcement ratio when compared to the wall section above 300 mm from the foundation. This increase in reinforcement ratio enhances the stiffness at this location and induces cracking in higher sections of the wall.

The curvature distribution and its evolution for the cycles parallel to the flange for 0.35%, 0.47% and 0.93% lateral drift are illustrated in Fig. 10. The average curvature was calculated along the entire height of the first floor of the specimen and is represented by a blue bar. The plots show that the average curvature at 0.47% drift was larger than the yield curvature estimated by Eq. 1. However, local measurements of the curvature distribution along the wall height revealed significant differences; for instance, the curvature during the web compression step exceeded that during the flange compression step. Additionally, the local curvatures are approximately twice the average, reaching values of 10/km and 7/km during the 0.93% lateral drift cycle for the web and flange compression, respectively. The distribution of localized curvatures (red bars) along the first floor during this loading step can be used to estimate the region where the reinforcement yielded. Figure 10 shows that at a 0.93% drift, the theoretical yield curvature (solid line) was exceeded for most sections along the lowest 925 mm of the wall, along both flanges and in both loading directions. For the load condition inducing compression on the web, the sensors on the west flange indicate that the yield curvature for 0.93% drift was also exceeded up to 1100 mm from the wall-foundation interface, covering almost the entire height of the first level of the specimen, up to the intermediate slab. It was not possible to define if the propagation of inelasticity continued above the slab as there was not localized instrumentation around that section.

Localized curvatures were not obtained for larger drifts because cracking propagation and concrete crushing resulted in the loss of anchorage of several sensors. However, based on the observed cracks at the end of the test, it is likely that the nonlinear response propagated further up both flanges and the web.

3.5 Deformation components

The contributions of shear, sliding and flexural deformation to the total displacement were assessed to explain the failure mode of the wall. Sliding deformation was directly measured from sensors at the wall-foundation interface, while flexure deformations were obtained using the rotations evaluated from the vertical displacement sensors located at the four corners of the wall. The shear deformations (Δ_s) were computed according to the method proposed by Haraishi (1984), which is defined according to Eq. 2.

$$\Delta_s = \frac{1}{4l_w^*} \left[(d + \delta_2)^2 - (d + \delta_1)^2 \right] - (\eta - 0.5)\theta(h_i)h_i \quad (2)$$

$$\eta = \frac{\int_0^{h_i} \theta(z) dz}{\theta(h_i)h_i} \quad (3)$$

where l_w^* is the width of the shear panel, d is the original diagonal length where the inclined sensors are installed in the panel, δ_1 and δ_2 are the elongation/shortening of the measured diagonal, $\theta(h_i)$ is the difference in the rotations at the top and bottom of the panel of height, and h_i and are measures of the variation in the curvature over the height of the panel.

The contributions of each of the components to the total displacement, as a fraction of the total displacement, are shown in Figs. 11 and 12. The flexure contribution was estimated using both the localized strains on the same floor and the average strains on the first floor. The former is represented in the plots in columns numbered 1, and the latter is represented in columns numbered 2.

The contributions of the deformation components for the flange parallel cycles (NS direction) are shown in Fig. 11 for lateral drift levels of 0.35%, 0.47% and 0.93% and for the instant of maximum displacement when these cycles impose compression on the web and for the instant when the cycles impose compression on the flange. The contributions were calculated individually based on the measurements of the sensors located at the east flange and at the west flange. In all cases, the flexure contribution is larger for the case of the average strain calculated with the average strains than that computed with the localized strains. This is probably related to unaccounted strains at each sensor location due to accidental movement of the anchorages of the sensors.

The shear-to- flexure displacement ratios (Δ_s/Δ_f) were obtained for the flange parallel cycles, resulting in average values from both flanges of 0.19, 0.25 and 0.28 for 0.35%, 0.47% and 0.93% lateral drifts, respectively, in the flange compressive direction. For the web compressive directions, the ratios obtained were 0.32, 0.45 and 0.35 for the same corresponding drifts. The larger Δ_s/Δ_f ratios for the web compression load direction compared to the flange compression direction coincide with previously reported trends by Beyer et al. (2011). The low variation of the Δ_s/Δ_f ratios as a function of lateral drift is also another trend reported in the same reference. However, the values of the Δ_s/Δ_f ratios obtained for the U-wall tested are larger than those reported by Beyer, probably in part due to the specific characteristics of the U-wall reported in this paper, such as its reduced

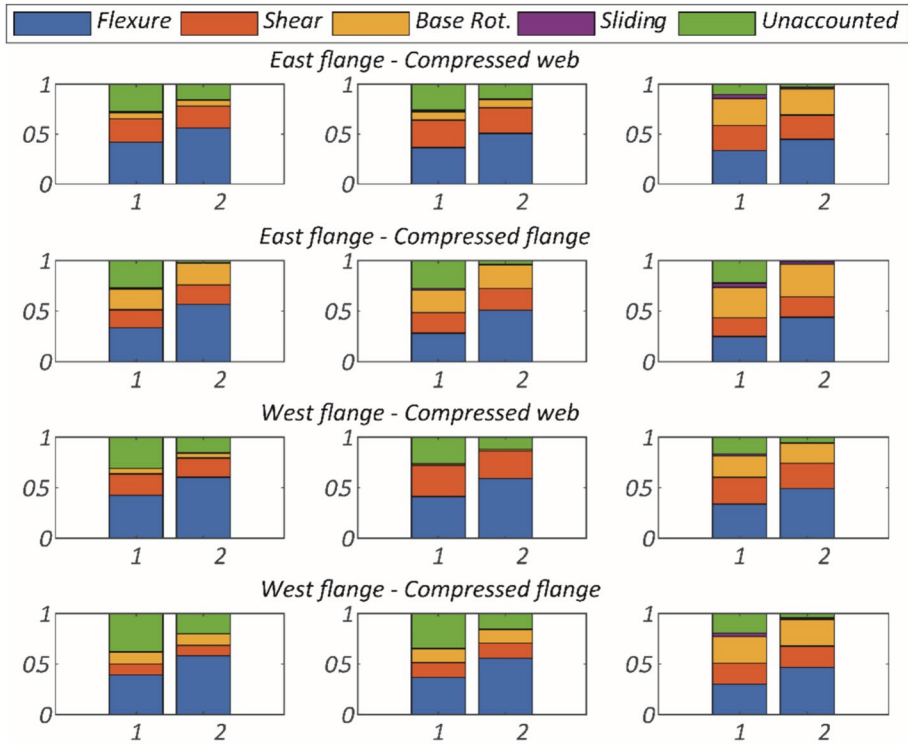


Fig. 11 Deformation components for flange parallel cycles (NS direction) at 0.35% (left column), 0.47% (middle column) and 0.93% drift (right column). For each plot, column 1 corresponds to the flexure contribution calculated from localized strains, and column 2 corresponds to the flexure contribution calculated from the average strains.

thickness and the absence of confined boundaries. Additional studies would have to be carried out to increase the data about this parameter.

For the web parallel cycles, the contributions were calculated for 0.35% and 0.47% lateral drifts (see Fig. 12) using the sensors located at the wall web. The contribution of the flexure component is also larger from the average strains than from the localized strains, and the shear deformation has a significantly larger contribution as the shear stresses in the web are significantly larger than the maximum stresses in the flanges. Sliding deformation is negligible, and the rotation contributions are not as large as those for the flange parallel cycles.

3.6 Equivalent plastic hinge length

The equivalent plastic hinge length is a key concept for structural performance-based design and there are several references with equations used to predict this parameter for C-shaped walls (Priestley et al 2007; Constantin 2016; Hoult et al 2018). The test data was used to estimate this parameter for the 0.93% cycle, which is the largest amplitude cycle with reliable information, as several sensors moved or detached from the wall due to cracking for larger amplitude cycles. The procedure to obtain the equivalent plastic

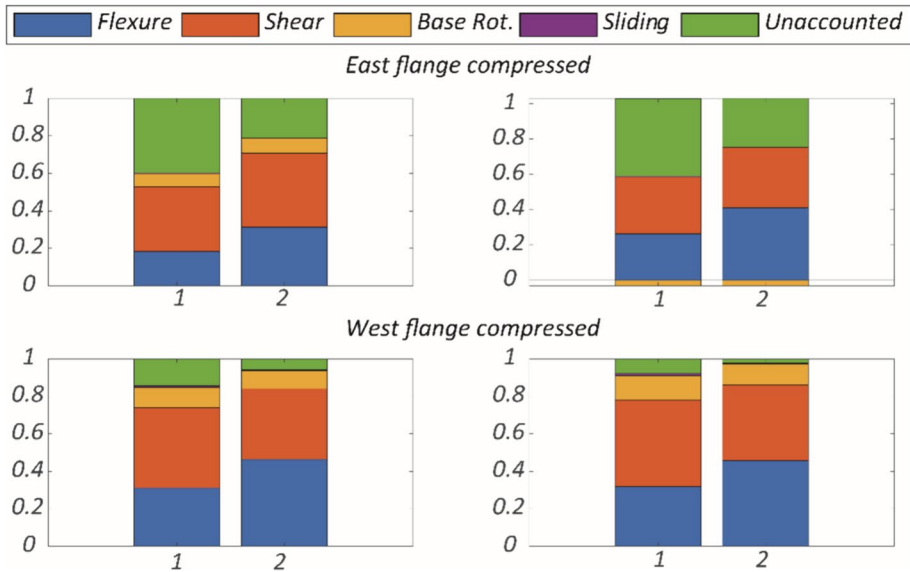


Fig. 12 Deformation components for web parallel cycles (EW direction) at 0.35% and 0.47% drift

hinge is described in Eq. 4–7, using a total displacement (Δ_t) of 22.6 mm for the 0.95% drift cycle. The yield displacement (Δ_y) was obtained as 6.45 mm, according to Eq. 6 from Priestley et al (2007) where ϕ_y is the yield curvature and equal to $2.45e^{-6}/\text{mm}$. H_e is the effective height of the cantilever wall (2620 mm) and H_n is the roof height of the cantilever wall, estimated as $H_n \approx H_e/0.7 \approx 3743\text{mm}$. The yield curvature was obtained according to Eq. 6 where K_l is a shape factor depending on the geometry of the section. A value of 1.4 is recommended for C-shaped walls. ϵ_{sy} is the expected yield strain for the steel (0.00228) and L_w is the wall dimension in the direction for which the displacement is being computed (1300 mm). The plastic hinge (L_p) was obtained from Eq. 7, based on the plastic displacement ($\Delta_p = 16.15\text{mm}$) and the maximum curvature (ϕ_u), which in this case was obtained using recordings from the sensors placed along the lowest 50 mm section of the wall (See Table 2). The equation considers the relative contribution of the shear deformations (Δ_s) with respect to the flexural deformations (Δ_f) (Hoult, 2022), which could be computed from the instrumentation. According to the instrumentation of the tested wall, the ratio (Δ_s/Δ_f) was computed as 0.36 for the case when the web was under compression and 0.28 for the case when the flange was in compression. Based on these results, the equivalent plastic hinge estimated was estimated as $L_p = 144\text{ mm}$ for the web compression cycles and $L_p = 112\text{ mm}$ for the flange compression cycles.

Table 2 Maximum recorded curvatures for the 0.93% drift cycles

$\phi_u = 3.4 * 10^{-5} / \text{mm}$	based on the maximum measured curvature along the first 50 mm at the bottom of the wall for compressed web cycle
$\phi_u = 4.5 * 10^{-5} / \text{mm}$	based on the maximum measured curvature along the first 50 mm at the bottom of the wall for compressed flange cycle

$$\Delta_r = \Delta_y + \Delta_p \quad (4)$$

$$\Delta_y = \frac{\emptyset_y}{2} H_e^2 \left(1 - \frac{H_e}{3H_n} \right) \quad (5)$$

$$\emptyset_y = \frac{K_1 \epsilon_{sy}}{L_w} \quad (6)$$

$$\Delta_p = L_p (\emptyset_u - \emptyset_y) H_e \left(1 + \frac{\Delta_s}{\Delta_f} \right) \quad (7)$$

The equivalent plastic hinge was also calculated using the common equation of $0.5 L_w$, which resulted in $L_p = 650$ mm. Eq. 8, recommended by Priestley et al (2007), was also used to obtain the plastic hinge length, where L_{sp} is the contribution of strain penetration to the plastic hinge, f_y is the yield strength of the reinforcement and d_b is the diameter of the reinforcement bars, results in $L_p = 431$ mm. Equation 9, proposed by Constantin (2016), results in a $L_p = 260$ mm for the flange compression cycle and $L_p = 215$ mm for the web compression cycle. In this equation τ is the average shear stress, which was computed using the maximum shear force during test and from the cross-section area, and ALR is the axial load ratio applied to the wall (4.4%). Finally, Eq. 1, which was proposed for lightly reinforced concrete U-shaped walls (Hoult, 2018), results in $L_p = 78$ mm.

$$L_p = KH_e + 0.1 L_w + L_{sp} \quad (8)$$

Where $L_{sp} = 0.0022 * 1.1 * f_y * d_b$

$$L_p = \left[0.05H_e + 0.05L_w \left(\frac{\tau}{0.17\sqrt{f'_c}} \right) \right] (1 + 4ALR) \quad (9)$$

$$L_p = (0.1L_w - 0.013H_e)(1 - 13ALR)(7e^{-0.8v}) \leq 0.5L_w \quad (10)$$

Where $v = \frac{\tau}{0.17\sqrt{f'_c}}$

Based on the equivalent plastic hinge calculated from sensor data and the proposed equations, it is evident that the experimental data aligns well with the proposed equations for U-shaped walls. Notably, commonly used equations, such as the $0.5 L_w$ simplification and the equation by Priestley (2007), tend to significantly overestimate the plastic hinge. The results are somewhere in the middle between the equation by Constantin (2016) and Hoult (2018).

3.7 Stiffness degradation

The stiffness deterioration was evaluated using the initial stiffness measured during the first load cycle at 0.04% drift in each loading direction as a reference. The tested specimen exhibited rapid stiffness degradation, as the stiffness decreased by 50% of the initial value

at only 0.18% lateral drift in the EW direction. For the NS direction, the same occurred for the web compression cycles. For flange compression cycles, the drift for this stiffness reduction was 0.3%, showing a slightly lower reduction rate compared to the opposite direction (see Fig. 13a). In the diagonal directions, the reduction also occurred rapidly at low drifts; however, the values of stiffness deterioration in Fig. 13b are not comparable to the values reported for the principal directions, as the load protocol considered applying diagonal cycles after yielding of the reinforcement during the flange parallel (AB) cycles and web parallel (CD) cycles.

4 Numerical modeling

A numerical model was calibrated to evaluate its accuracy in estimating key engineering parameters simultaneously at the global and local levels, which has been a long-established challenge for the earthquake engineering community. The beam-truss type model approach has been successfully used for different types of walls, including U-shaped walls (Lu and Panagiotou 2014), and it is commonly used in both engineering design and assessment practice (Kolozvari et al. 2018; Arteta et al. 2019). For thin walls, several studies have modeled these elements using various techniques to capture their characteristic features, such as the use of a single layer of reinforcement, out-of-plane response, and low-ductility steel (Alvarez et al., 2020; Arroyo et al. 2021; Rosso et al. 2022; Carrillo et al. 2022). The studies by Alvarez et al. (2020) and Carrillo et al. (2022) use the beam-truss approach and obtained satisfactory results for the global response. The same approach based on beam-truss models was implemented in Seismostruct (2016) U-shaped cross-sectional geometry, obtaining a high degree of accuracy (Hoult et al 2023a, b).

The model consists of displacement-based frame elements to represent vertical and horizontal steel-rebar reinforced sections and concrete-only diagonal truss elements, as shown in Fig. 14. The steel reinforcement of the wall was distributed among the 18 vertical elements, and the 13 horizontal elements characterized the steel as closely as possible to the transverse section, as shown in Fig. 2b. As there are no confined section regions in the wall, the unconfined concrete model by Kent, Scott and Park (1971), together with

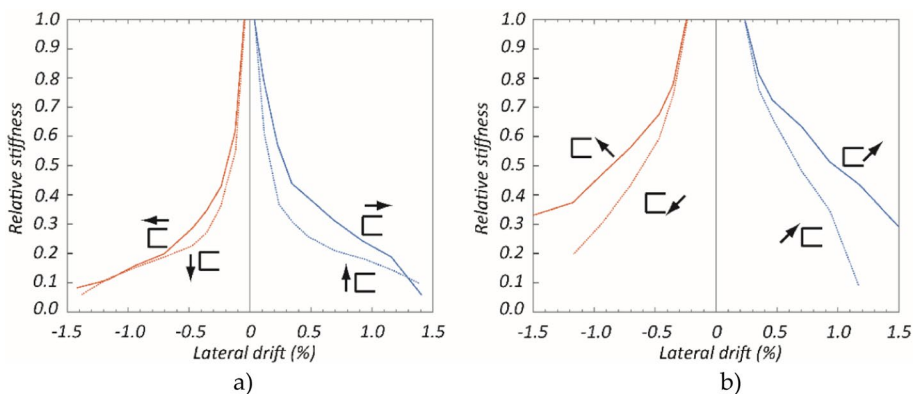


Fig. 13 Stiffness degradation under cyclic loading. **a** web parallel and flange parallel cycles. **b** diagonal cycles

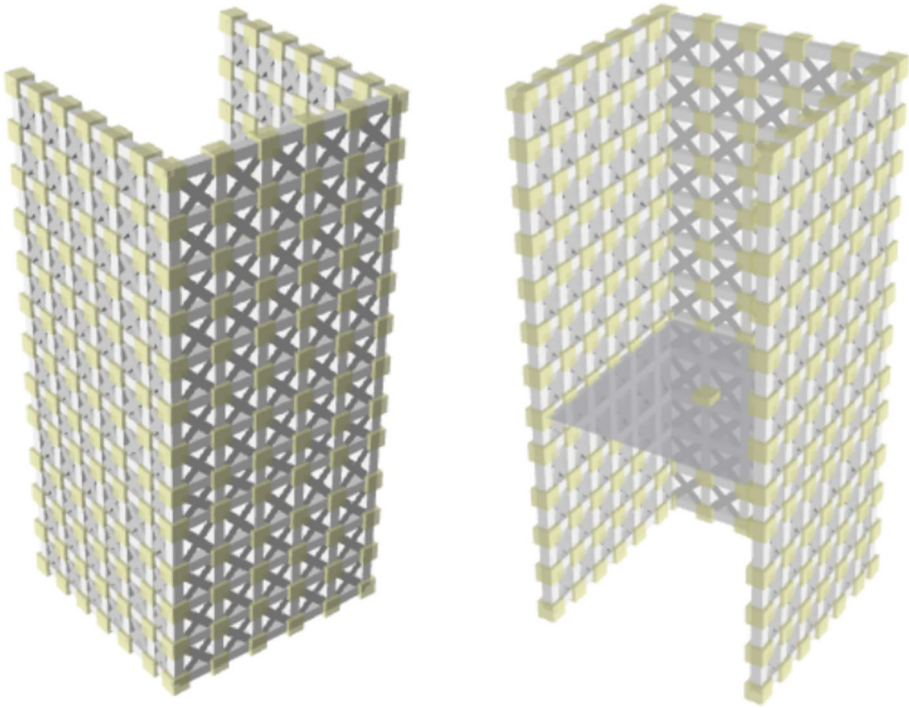


Fig. 14 Beam truss model of the tested wall

cyclic rules according to Karsan and Jirsa (1969), was used for the frames and the truss elements. The steel was represented with the model proposed by Giuffrè-Menegotto-Pinto (Filippou et al. 1983). The material properties were defined using the measured values (see Sect. 2.1). The element geometries, namely, the vertical and horizontal beam elements and the diagonal truss elements, were defined following the recommendations of Lu, Panagiotou and Koutromanos (2014). Due to the differences in the reinforcement distribution, horizontal steel ratios and shear strength between the direction parallel to the flanges and the direction parallel to the web, the geometry was adjusted to achieve a strut angle (θ_d) of approximately 50° for the flanges and close to 45° for the web, according to the same recommendations and described in Eq. 2.

$$\theta_d = \tan^{-1} \left(\frac{v_{max}}{f_{yt} \rho_t t_w d} \right) \leq 65^\circ \quad (11)$$

where v_{max} is the maximum resisted lateral force, f_{yt} is the yield strength of the transverse reinforcement, ρ_t is the transverse reinforcement ratio, t_w is the thickness of the wall and d is the distance between the outer vertical edges of the model in the direction of loading.

The intermediate slab was included using a rigid diaphragm constraint, the effect of the top slab was introduced by applying the same displacement history to all nodes at the top, and the foundation was represented by fixing the nodes at the base. A static time-history analysis was carried out, applying the entire multidirectional loading protocol. This model does not include other more refined details, such as the reduction of the strut capacity in terms of the axial strut deformation or buckling of the steel.

The lateral force versus displacement obtained from the numerical model and the experimental test for all the directions are shown in Fig. 15. In general, the estimation from the numerical model is consistent with the experimental results in terms of the maximum force for all directions. The numerical model stopped when it was not possible to reach convergence during a flange parallel cycle (NS direction) at a displacement similar to the actual failure of the test unit (see Fig. 15a) due to the crushing of the flange bottom edges. In the experimental tests, cycles reached larger displacements as the load protocol continued during the test to observe other failure modes; however, these were not represented by the numerical model.

The numerical model also showed significant differences for the initial cycles of the diagonal loading stages for the direction that induced compression at the flanges. Figure 15c and d show that for these diagonal loads, the model overestimated the capacity in both the SW–NE and SE–NW directions.

The model tends to underestimate the initial stiffness of the wall, as the experimental-to-numerical ratio in Fig. 16 shows values larger than 1 for the initial cycles. This could be related to the type of concrete model used because it has no tensile capacity. Other material options available in the library of the software were used in previous configurations of the model to include the tensile strength of the concrete for the vertical and horizontal beam elements. When using the tensile concrete strength, the match between

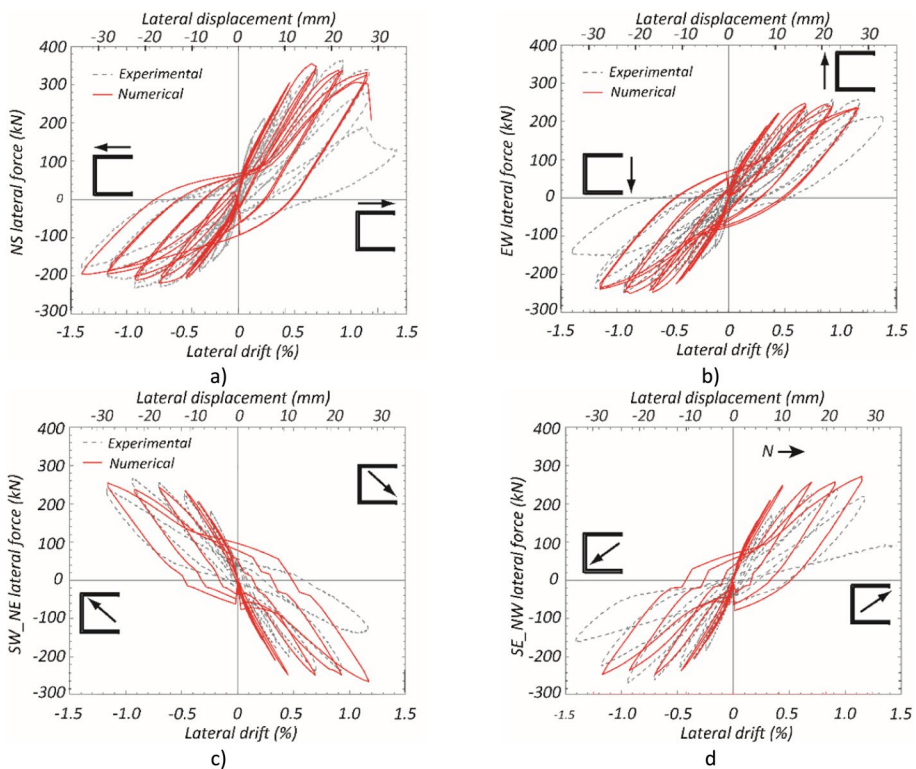


Fig. 15 Experimental vs numerical global comparison for the u-shaped wall tested. **a** hysteresis cycles for loading parallel to the flanges. **b** hysteresis cycles for loading parallel to the web. **c** hysteresis cycles for diagonal loading in the SW_NE direction. **d** hysteresis cycles for diagonal loading in the SE_NW direction

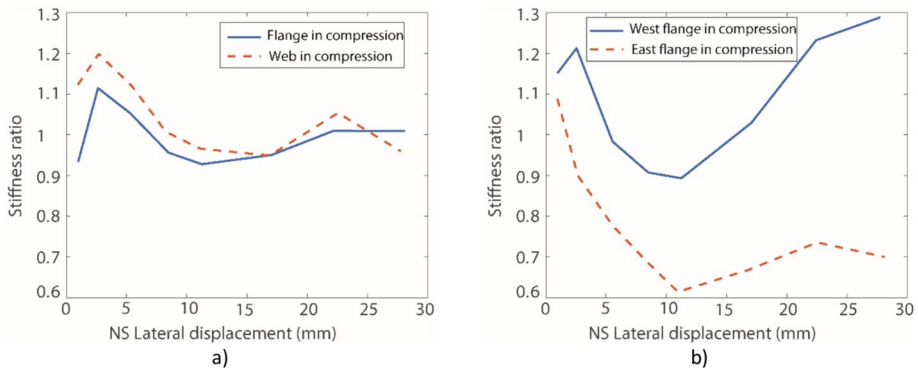


Fig. 16 experimental to numerical stiffness ratio. **a** Loading cycles parallel to the flanges (NS direction). **b** Loading cycles parallel to the web (EW direction)

the experimental and numerical hysteresis response is slightly improved for these initial cycles; however, when using a concrete with a trilinear envelope with no confinement and with tensile strength capacity, it was not possible to reach convergence for drifts larger than 0.5%. A second alternative, which used a Mander type of concrete with no confinement (Mander et al 1988), did not manage to capture the failure of the wall that occurred due to the crushing of the concrete at the flange toes. Additionally, a significantly larger displacement capacity was estimated from that option compared to the final alternative used, as the Mander concrete model has a strain capacity larger than that of concrete models that do not have the alternative of modeling confinement.

From a global point of view, for the numerical hysteresis cycles for the cycles below 0.96% lateral drift, corresponding to 22 mm in the flange parallel loading (NS direction), the energy dissipated is lower than the energy dissipated experimentally (see Fig. 17a), and the same occurs in the web parallel loading cycles (EW direction) for lateral drifts below 0.7%, corresponding to 18 mm of lateral displacement. For larger drifts, the model overestimates the experimental energy dissipation. However, for the web parallel cycles, the maximum overestimation was only 13%, which occurred at the instant of failure, while for the web parallel cycle, the maximum overestimation was 35%.

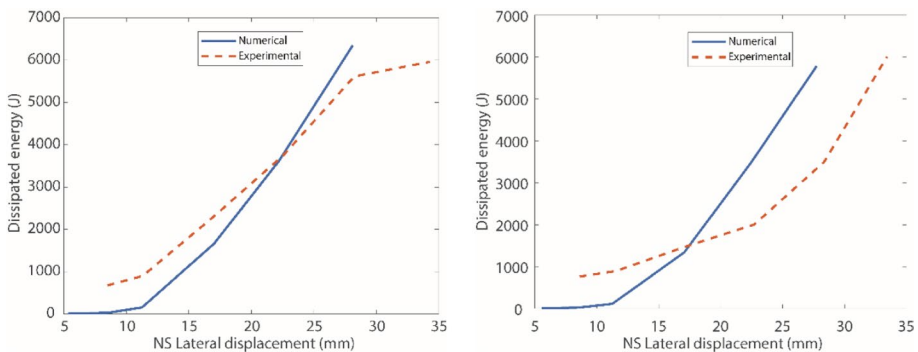


Fig. 17 Experimental vs numerical comparison of the dissipated energy

An attempt was carried out to calibrate fiber-based models with the experimental results for the flange parallel cycles. The models consisted of a cantilever stick and a U-shaped transverse section with 600 fibers. The first model used a single force-based (FB) element, while the second model used displacement-based (DB) elements (see Fig. 18a). Three integration points were used for the FB element model, and the length of the element at the base of the wall, for the DB element model, was defined as 450 mm. The values were obtained from an iterative procedure to achieve the best possible match to the experimental envelope.

A Hilber-Hughes-Taylor Integration scheme was employed, and the same load protocol and software were used for the analysis. Both models utilized the same steel material as the beam-truss model. For the concrete, a Mander model without confinement was used, as the concrete model from the beam-truss model did not provide accurate results. The model using DB elements (short dashed line) showed a better match to the experimental results (long dashed line) than the model using FB elements (solid line) (See Fig. 18b).

Although the DB element model provided relatively accurate results for the maximum forces in both directions, the fiber-based models significantly overestimated the energy dissipated, initial stiffness and forces for the initial cycles.

For the analysis at the local level, the strains at the web west (WW), web east (WE) and flange east (FE) edges of the wall were compared between the experimental test (_E) and numerical model (_M) results. The flange west (FW) was not included because the sensors malfunctioned. The reported strains correspond to the bottom 230 mm, which is the length of the vertical beam elements of the model at these locations. The displacements recorded from the sensors along that section were used to compute the average strain. Initially, for the loading parallel to the flanges (NS direction), there is relatively good agreement of the numerical estimation of the tensile strains for both the flanges and web edges as the relative error decreases as the strain and lateral displacement increase (see Fig. 19a). The compressive strains at the web are underestimated by the model, which in part could be

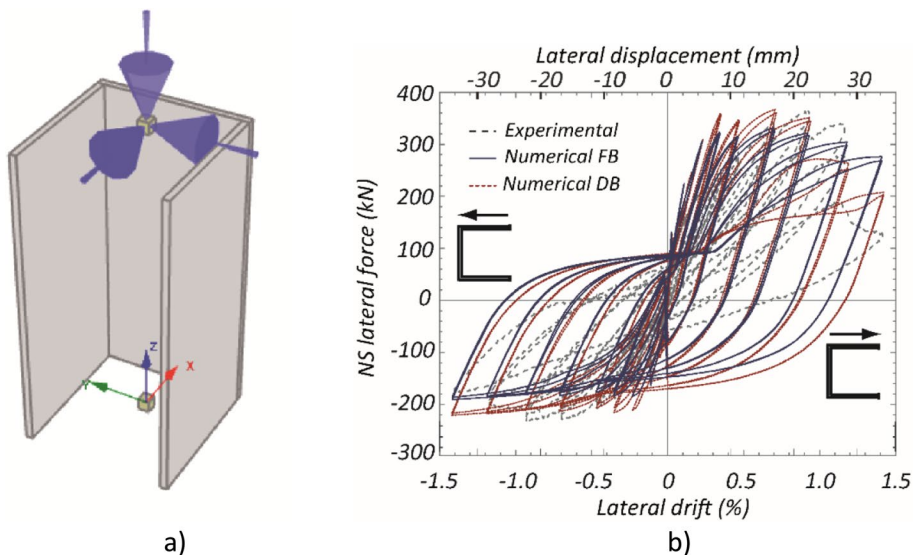


Fig. 18 Fiber model of the U-shaped tested wall (right) and comparison of the experimental vs. numerical global force vs. displacement in the flange parallel direction

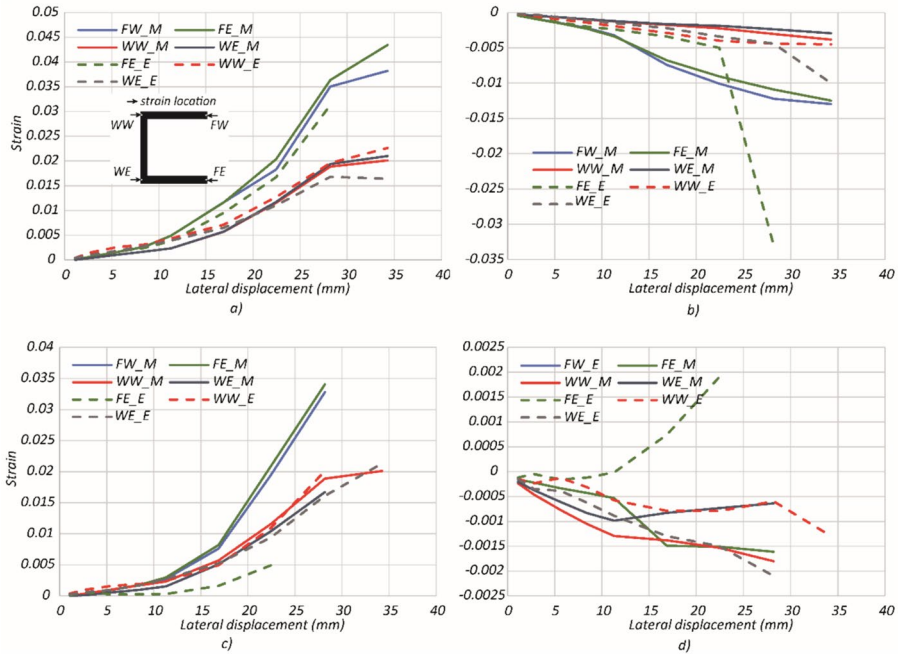


Fig. 19 Experimental vs numerical local comparison of the strain envelope at the four bottom edges of the U-shaped tested wall. **a** tensile strains for the loading parallel to the flanges. **b** compressive strains for the loading parallel to the flanges. **c** tensile strains for the loading parallel to the web. **d** compressive strains for the loading parallel to the web

due to the location where the strains are obtained from the model. The results reported by the software are located at the centroid of one of the fibers, which is approximately 5 mm inside the concrete, while for the test, the strains are located at the location of the potentiometers, which was approximately 10 mm from the wall surface. As the neutral axis depth is shallow when the web is in compression, this slight location difference may have an appreciable impact. For the flanges, the error is even larger, and the numerical model fails to estimate the behavior obtained from the test (see Fig. 19b).

For the cycles parallel to the web, there is a good match between the experimental and numerical results for the tensile strains in the web; however, for the flanges, the experimental strains measured are significantly smaller than the numerical strains (see Fig. 19c). For compressive strains in this same direction, the numerical model does not match the experimental strains (see Fig. 19d), as it has been the case for similar numerical models previously reported by Hoult (2023a).

5 Conclusions

Reinforced concrete (RC) structural walls with nonrectangular geometries are widely used as part of structural systems in seismic prone areas, and in some regions, these walls have particular characteristics such as thicknesses between 100 and 150 mm, no boundary confined elements and fragile electrowelded wire mesh as part of the vertical and transverse

reinforcement. This document presents experimental and numerical findings of a half-scale U-shaped thin concrete wall reinforced with electrowelded wire mesh and subjected to multidirectional loading.

The experimental results indicated that the 4 mm wires at the foundation-wall interface fractured due to drifts close to 1%, and there was buckling of the bars at the flange toes. Despite the evidence of significant cracking, the electrowelded steel did not rupture. The displacement capacity of the wall was significantly limited due to the failure associated with concrete crushing at the flange toes for the 1.18% drift cycles. Based on the limited drift and the impossibility of adding confinement to the edges due to geometrical constraints, these types of walls would only be feasible in seismic regions if the strains at the flanges are below the crushing strains, which would require that the drifts under the seismic demands are kept low by using a stiff structural system with a high density of walls.

Loading in the diagonal direction resulted in fragile failure for a cycle inducing compression on a flange toe at 1.4% lateral drift; however, this direction did not result in the failure of the wall, as occurred during a previous drift for flange parallel loading, when the cycles induced compression in the flange toes.

The strain distribution indicates that inelasticity propagates primarily along the lowest 900 mm section of the first level of the element. The largest deformations occurred at the foundation-wall interface and approximately 400 mm above this level, corresponding to the location where the 4 mm reinforcement from the foundation, splicing the electrowelded wire mesh, was fractured. As fracture at the wall-foundation occurred, it would be recommended to increase the steel ratio along the flanges to increase the spread of cracking and improve the wall performance by preventing steel fracture and crack concentration. Further research will be needed to evaluate the effect of the reinforcement ratio on the displacement capacity for this type of element.

The stiffness decreased significantly at moderate drifts, reaching values of 30–40% of the initial stiffness at a 0.5% lateral drift and 15–20% of the initial stiffness at approximately 1% drift. Based on these values, it is recommended to use stiffness reduction factors below 0.4 to adjust the gross element stiffness when computing drifts from elastic numerical models in practical designs of buildings with thin U-shaped walls.

Numerical models based on the assumption of plane sections do not provide accurate estimates of the multidirectional response of walls. The use of beam-truss models significantly improved the estimation of the global response of the wall, even though features such as the compressive strut capacity reduction as a function of the strut axial strain and the buckling of steel were not considered in the model. Stiffness variation and energy dissipation were also estimated with an acceptable level of accuracy, showing significantly better results for flange-parallel than those for web-parallel cycles.

At the local level, the estimation of tensile strains is also estimated within an acceptable range of accuracy for most of the cases, but the estimation of the concrete strains is more difficult.

The dataset on thin U-Shaped walls is still limited and further studies will be required to increase the information about some relevant aspects for these walls such as the effect of the slab on the propagation of cracking, the contribution of shear deformations and the minimum steel ratio to control bar fracture.

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Declarations

Conflict of interest The authors have no relevant financial or nonfinancial interests to disclose. The datasets generated during and/or analyzed during the current study are available from the corresponding author upon reasonable request.

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References

- Almeida J, Prodan O, Rosso A, Beyer K (2017) Tests on thin reinforced concrete walls subjected to in-plane and out-of-plane cyclic loading. *Earthq Spectra* 33(1):323–345
- Álvarez R, Restrepo JJ, Panagiotou M (2020) RC wall plastic hinge out-of-plane buckling: analysis using the nonlinear beam-truss model. *J Struct Eng* 146(12):04020274
- Arroyo O, Feliciano D, Carrillo J, Hube MA (2021) Seismic performance of mid-rise thin concrete wall buildings lightly reinforced with deformed bars or welded wire mesh. *Eng Struct* 241:112455
- Arteta CA, Moehle JP (2023) Compressive behavior of thin rectangular boundary elements. *ACI Struct J* 120(2):157–170. <https://doi.org/10.14359/51737236>
- Arteta CA, Araújo GA, Torregroza AM, Martínez AF, Lu Y (2019) Hybrid approach for simulating shear-flexure interaction in RC walls with nonlinear truss and fiber models. *Bull Earthq Eng* 17(12):6437–6462. <https://doi.org/10.1007/s10518-019-00681-6>
- AIS (2010) Reglamento colombiano de construcción sismo resistente NSR-10 Tomo 2. Bogota, Colombia: Asociación Colombiana de Ingeniería sísmica
- Behrouzi AA, Mock AW, Lehman DE, Lowes LN, Kuchma DA (2020) Impact of bidirectional loading on the seismic performance of U-shaped piers of core walls. *Eng Struct* 225:111289
- Behrouzi A, Mock A, Lehman D, Lowes L and Kuchma D (2018) Seismic performance of slender U-shaped walls subjected to Uni- and -bidirectional loading. In: 11th U.S. National Conference on Earthquake Engineering, Integrating Science, Engineering & Policy, June 25–29, Los Angeles, California
- Beyer K, Dazio A, Priestley MJN (2008) Quasi-static cyclic tests of two U-shaped reinforced concrete walls. *J Earthq Eng* 12(7):1023–1053. <https://doi.org/10.1080/13632460802003272>
- Beyer K, Dazio A, Priestley N (2011) Shear deformations of slender reinforced concrete walls under seismic loading. *ACI Struct J* 108(2):167–177
- Blandon CA, Bonett R (2020) Thin slender concrete rectangular walls in moderate seismic regions with a single reinforcement layer. *J Build Eng* 28:101035. <https://doi.org/10.1016/j.jobbe.2019.101035>
- Blandon C, Arteta C, Bonett R, Carrillo J, Beyer K, Almeida J (2018) Response of thin lightly reinforced concrete walls under cycling loading. *Eng Struct* 176:175–187. <https://doi.org/10.1016/j.engstruct.2018.08.089>
- Bonett RL, Carrillo J, Blandon CA, Arteta CA (2024) Understanding the behavior of the thin lightly reinforced concrete wall (TLRCW) building system. *J Struct Integr Maint TSTR*. <https://doi.org/10.1080/24705314.2024.2337449>

- Brueggen BL, French CE, Sriharan S (2017) T-shaped RC structural walls subjected to multidirectional loading: test results and design recommendations. *J Struct Eng* 143(7):04017040
- Carrillo J, Diaz C, Arteta CA (2019) Tensile mechanical properties of the electrowelded wire meshes available in Bogotá, Colombia. *Constr Build Mater* 195:352–362. <https://doi.org/10.1016/j.conbuildmat.2018.11.096>
- Carrillo J, Cubillos E, Parra PF (2022) Modeling the seismic response of thin concrete walls using the non-linear beam-truss model. *J Build Eng* 52:104424. <https://doi.org/10.1016/j.jobbe.2022.104424>
- Constantin R, Beyer K (2016) Behavior of U-shaped RC walls under quasistatic cyclic diagonal loading. *Eng Struct* 106:36–52
- Constantin R (2016). Seismic behaviour and analysis of U-shaped RC walls, in À la faculté de l'environnement naturel, architectural et construit. École polytechnique fédérale de Lausanne: Lausanne, Switzerland.
- Filippou FC, Popov EP, Bertero VV (1983) Effects of bond deterioration on hysteretic behavior of reinforced concrete joints. Report EERC 83–19, Earthquake Engineering Research Center, University of California, Berkeley.
- Hiraishi H (1984) Evaluation of shear and flexural deformations of flexural type shear walls. *Bull N Z Soc Earthq Eng* 17(2):135–144
- Hoult R, Goldsworthy HM, Lumantarna E (2018) Plastic hinge length for lightly reinforced c-shaped concrete walls. *J Earthq Eng* 2018:1–32
- Hoult R, Appelle A, Almeida J, Beyer K (2020) Seismic performance of slender RC U-shaped walls with a single-layer of reinforcement. *Eng Struct* 225:111257
- Hoult R, António AC, Almeida J (2023a) Beam-truss models to simulate the axial-flexural-torsional performance of RC U-shaped wall buildings. *CivilEng* 4(1):292–310. <https://doi.org/10.3390/civileng4010017>
- Hoult R, Doneux C, Pacheco de Almeida J (2023b) Tests on reinforced concrete U-shaped walls subjected to torsion and flexure. *Earthq Spectra* 39(4):2685–2710
- Ile N, Reynouard JM (2005) Behavior of U-shaped walls subjected to uniaxial and biaxial cyclic lateral loading. *J Earthq Eng* 9(1):67–94. <https://doi.org/10.1080/13632460509350534>
- Karsan ID, Jirsa JO (1969) Behavior of concrete under compressive loading. *J Struct Division* 95(12):2543–2564
- Kent DC, Park R (1971) Flexural members with confined concrete. *J Struct Division* 97(7):1969–1990
- Kolozvari K, Arteta CA, Fischinger M, Gavridou S, Hube M, Isakovic T, Lowes LN, Orakcal K, Vásquez J, Wallace J (2018) Comparative study of state-of-the-art macroscopic models for planar reinforced concrete walls. *ACI Struct J* 115(6):1637–1657. <https://doi.org/10.14359/51710835>
- Lu Y, Panagiotou M (2014) Three-dimensional nonlinear cyclic beam-truss model for reinforced concrete non-planar walls. *ASCE J Struct Eng*. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0000852](https://doi.org/10.1061/(ASCE)ST.1943-541X.0000852)
- Mander JB, Priestley MJN, Park R (1988) Theoretical stress-strain model for confined concrete. *J Struct Eng* 114(8):1804–1826
- Miranda-Giraldo M, González-Olier C, Zambrano HR, Arteta CA (2024) Characterization of the micro-structure and mechanical behavior of electrowelded wire meshes: a case study in Colombia. *Case Stud Constr Mater* 20:e02886. <https://doi.org/10.1016/j.cscm.2024.e02886>
- Ortega R, Torres P, Thomson P, Marulanda J, Areiza G (2023) Behavior under lateral cyclical load of thin reinforced concrete walls of the industrialized system. *Engineering Structures* 294:116634. <https://doi.org/10.1016/j.engstruct.2023.116634>
- Priestley MJN, Calvi GM, Kowalsky MJ (2007) Displacement-based seismic design of structures. IUSS Press, Pavia
- Rosso A, Almeida JP, Beyer K et al (2016) Stability of thin reinforced concrete walls under cyclic loads: state-of-the-art and new experimental findings. *Bull Earthq Eng* 14:455–484
- Rosso A, Jiménez-Roa LA, Almeida JPD, Beyer K (2022) Instability of thin concrete walls with a single layer of reinforcement under cyclic loading: numerical simulation and improved equivalent boundary element model for assessment. *J Earthq Eng* 26(1):493–552
- Sullivan TJ, Priestley MJN, Calvi GM (2012) Model code for the displacement-based seismic design of structures. IUSS Press, Pavia
- Sutton MA, Ortu JJ, Schreier HW (2009) Image correlation for shape, motion and deformation measurements: basic concepts, theory and applications New York. Springer, NY

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