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EXPERIMENTAL STUDY ON SEISMIC BEHAVIOR OF FIBER–REINFORCED CONCRETE STRUCTURAL WALLS FOR LOW–RISE BUILDINGS

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Abstract

Industrialized reinforced concrete (RC) systems have become a solution widely used in some Latin American countries as an efficient alternative system for housing construction, compared with traditional masonry infilled frames and reinforced masonry systems. Industrialized systems provide high efficiency in construction through an appropriate planning of activities, budget, staff, equipment and materials, generating a fast and effective mass production process. The project intends to validate the behavior of fiber–reinforced concrete structural walls, in order to implement this solution for industrialized systems. The project aims at optimizing the construction time and reinforcement layout, and improving its efficiency without affecting the durability, serviceability and seismic performance of low–rise buildings. As a part of this project, an extensive literature revision process has been conducted on construction and design experience of industrialized RC systems in Colombia and the experimental behavior of RC structural walls under seismic loads. The state–of–the–art of high performance fiber–reinforced concrete has been studied as well, as part of this revision. Several prototypes of low–rise buildings were selected and analyzed for low, intermediate and high seismic hazard scenarios, in order to determine seismic demand characteristics acting on typical structural walls. According to those results, a series of full–scale experimental tests of isolated RC structural walls were designed and planned to be subjected to in–plane quasi–static cyclic loads. The experimental program includes quasi–static test of eight types of walls. The parameters evaluated are the type of high performance concrete mixture (volume fraction of steel fibers), and flexural reinforcement. The paper discusses the effect of seismic demands over low–rise buildings comprised of RC shear walls, and the preliminary results of the first three walls of the experimental program such as the measured performance of structural walls in terms of failure modes, hysteresis curves, and contribution of fiber to shear strength.

Keywords: high performance concrete, structural walls, low–rise buildings.

1. Introduction

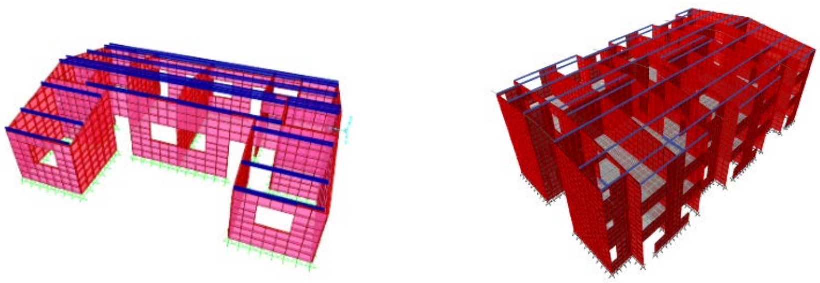
Traditionally, affordable housing construction in Latin American countries has been based on clay masonry walls, whether it is unreinforced masonry or as masonry infilled concrete frames. This trend has occurred mostly due to the availability of construction materials, lack of specialization and technical level of labor, and therefore low labor cost. However, further development of this type of construction has been practically inexistent, and it still remains as a handmade process. Additionally, self–constructed masonry houses (which are very common) fail to have proper detailing for seismic loads, thus enhancing their vulnerability. Industrialized reinforced concrete (RC) systems have emerged as a solution to solve some of the clear disadvantages of masonry wall structures. These solutions are based on manageable and practical formwork for cast–in–place walls and slabs, and in most cases, the formwork allows placing the concrete of an entire apartment or story as a monolithical block. Therefore, such structural system is comprised of numerous interior walls in at least the two orthogonal plan directions. The use of industrialized RC walls for affordable housing has led to minimize construction costs tightly by using minimum wall thicknesses and only one layer of steel mesh as web shear reinforcement. The latter is achieved since demands on low–rise buildings are low, and the design of this systems is controlled by minimum code provisions. Nevertheless, the direct construction cost of these systems may be reduced even more if steel reinforcement is reduced accordingly to the demand, and reducing the labor cost of placing this reinforcement as well. Fiber–reinforced concrete (FRC) may be considered as a feasible alternative, by using it as a replacement of the reinforcement in the wall web.

FRC is a composite material characterized by a cementitious matrix and discrete fiber reinforcement. Its main advantage is the capability of developing residual tensile strength after cracking [1]. In such post–cracking behavior, steel or synthetic fibers start acting by sewing the cracks, and developing such residual tensile strength due to the slip of fibers from the concrete matrix. The expected slippage mode of failure is ductile since it allows energy dissipation by both the appearance of multiple small cracks rather than single wide cracking, and controlling the tensile rupture of fibers. However, a sudden loss of loading capacity may be expected for low fiber dosages, whereas for a higher content of fibers in the matrix, residual tensile strength is achieved by strain hardening, depending on the geometric and mechanical properties of fibers [2]. At fiber dosages over 1% in concrete volume fraction, a strain hardening mechanism is expected, and a post–cracking behavior with residual strength higher than the cracking strength of concrete is observed. On the other hand, contents below 1% of concrete volume fraction, a rather fragile behavior is expected, showing residual strength lower than the cracking strength of concrete [3]. Depending on the demand level, one of the latter may be selected as appropriate.

In order to explore the applicability of FRC walls as an alternative for industrialized RC systems in housing construction in Colombia, the Research Center on Materials and Civil Infrastructure of the Universidad de los Andes, in Bogotá, is conducting a research project entitled "Technical Validation of High Performance Concrete Structural Walls for Low–Rise Buildings". As part of this project, an extensive literature revision process has been conducted on construction and design experience of industrialized RC systems in some Latin American countries such in Colombia, Mexico and Peru, and the observed behavior of RC structural walls under seismic loads. The state–of–the–art of high performance fiber–reinforced concrete has been studied as well, as part of this revision. Several prototypes of low–rise buildings were selected and analyzed for low, intermediate and high seismic hazard conditions, in order to determine seismic demand characteristics acting on typical structural walls. This analysis was conducted using nonlinear time history procedures. According to these results, a series of full–scale experimental tests of isolated reinforced concrete structural walls were designed and planned to be subjected to in–plane quasi–static reversed–cyclic loads. The experimental program included quasi–static test of eight types of walls. The parameters evaluated are the type of high performance concrete mixture (volume fraction of steel fibers), and the flexural reinforcement considered. This paper shows an overview of the ongoing research project in terms of the assessment of seismic demands in two low–rise building prototypes comprised of RC shear walls, and the results of the first three walls of the experimental program for structural wall specimens (failure modes, hysteresis curves, and contribution of fibers to shear strength).

2. Seismic demands on low–rise buildings

Two distinct building prototypes (one and three stories) were selected for evaluating the seismic demands on structural walls for low–rise buildings. The buildings were analyzed according to the code provisions from the Colombian Building Code (NSR–10) [4], which are based on ACI 318–08 [5]. The prototypes selected are RC buildings currently constructed in Colombia. Both prototypes are comprised of 80 mm–thick RC walls, and the depth of floor slabs is 100 mm. Light roof systems for both prototypes consisted on cold–formed steel beams for supporting the roof sheathing. Building prototypes are showed in Fig. 1.



(a) (b)

Fig. 1 – Building prototypes: (a) one–story building; (b) three–story building

2.1. Seismic hazard levels and acceleration records

Three different seismic hazard levels were used for evaluating the demand on structural walls; such hazard levels are related to the three seismic hazard regions established by the NSR–10 Colombian Code. Therefore, the main cities in each region (based on population and construction indexes) were selected¡, namely Barranquilla for low hazard, Bogotá for intermediate hazard, and Cali for high hazard. Fig. 2 shows the design spectra for these cities, considering site soil conditions for areas in the cities where affordable housing would probably be constructed based on city expansion and development.

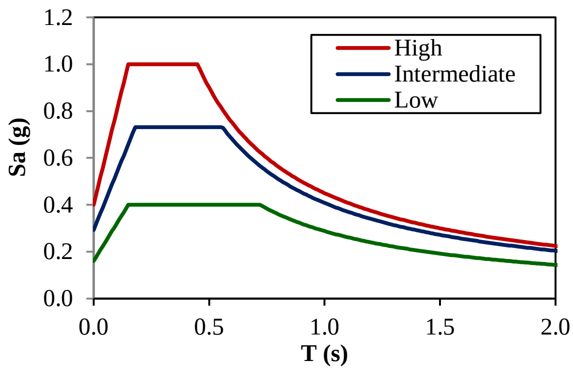
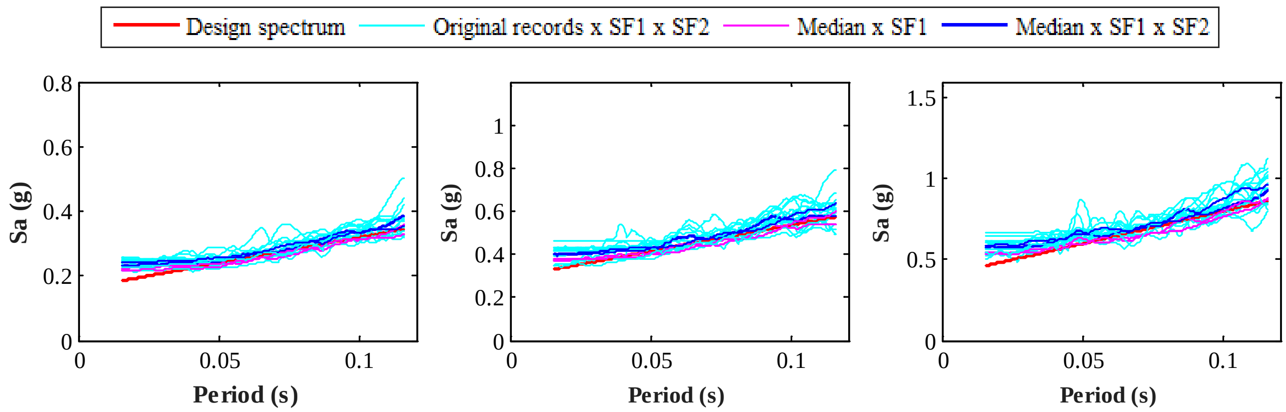


Fig. 2 – Design spectra for low, intermediate and high seismic hazard levels

A nonlinear time history analysis was conducted for these prototypes. Acceleration records were scaled according to each design spectrum. Records were selected from the PEER Ground Motion Database (http://ngawest2.berkeley.edu) according to seismic deaggregation procedures for the specific site conditions considered. Seven records for each hazard level (with two orthogonal directions each) were selected and scaled by means of the scaling procedure defined by the ASCE/SEI 7–10 [6]. Scaling results are presented in Fig. 3 for the three hazard levels and the range of periods considered. Fundamental periods for scaling were obtained from linear elastic models of the building prototypes using SAP2000, developed as part of the larger research project, but outside the scope of this paper.

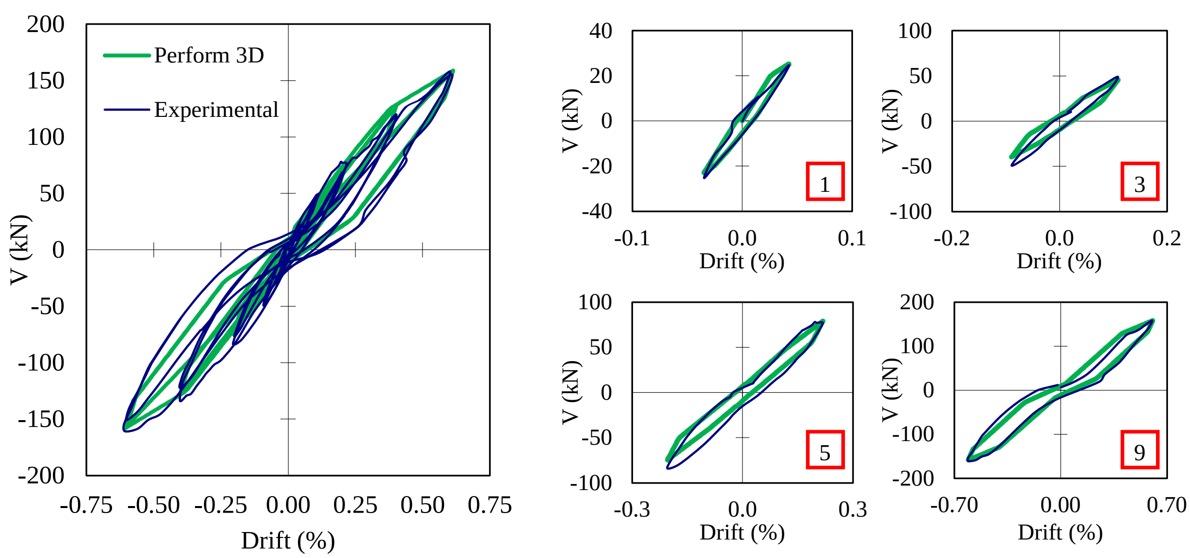


(a) (b) (c)

Fig. 3 – Scaled acceleration records for three hazard levels: (a) low; (b) intermediate; (c) high

2.2. Model definition and calibration

Prototypes were modeled and analyzed using Perform3D for estimating the inelastic demands acting on walls. Structural RC walls were modeled as general wall elements, and material properties were based on the original design of the actual buildings (f'_c = 21 MPa, E_c = 17872 MPa, f_{y_mesh} = 630 MPa, E_s = 200000 MPa). The estimated fundamental vibration periods for the one– and three–story buildings were 0.08 s and 0.09 s, respectively. The calibration of the general wall model, having both shear material and compression diagonal elements, was conducted using experimental data from a previous study [7]. Fig. 4 shows the comparison between the experimental data and the calibrated model. The model defined is able to represent adequately loading and displacement capacity, area enclosed by cycles, secant stiffness, and pinching behavior.



(a) (b)

Fig. 4 – General wall model calibration: (a) comparison with experimental data; (b) comparison per main cycles

2.3. Results of nonlinear time history analysis

Results of the nonlinear numerical models are presented in terms of drifts and shear stresses acting on walls. To quantify the relative magnitude of such demands, drifts (Fig. 5) are compared to performance limits (IO, LS, CP) proposed by Carrillo and Alcocer [7] for wire–mesh RC walls. Shear stresses (Fig. 6) are normalized with respect to the concrete shear strength (τ_{cr}). Results are presented for the three hazard levels and for the ten most demanded piers in the model. As expected, drifts and shear stresses demands acting on these type of rigid systems are considerably low compared to prescribed limits in most building codes, and therefore the design is usually controlled by minimum provisions.

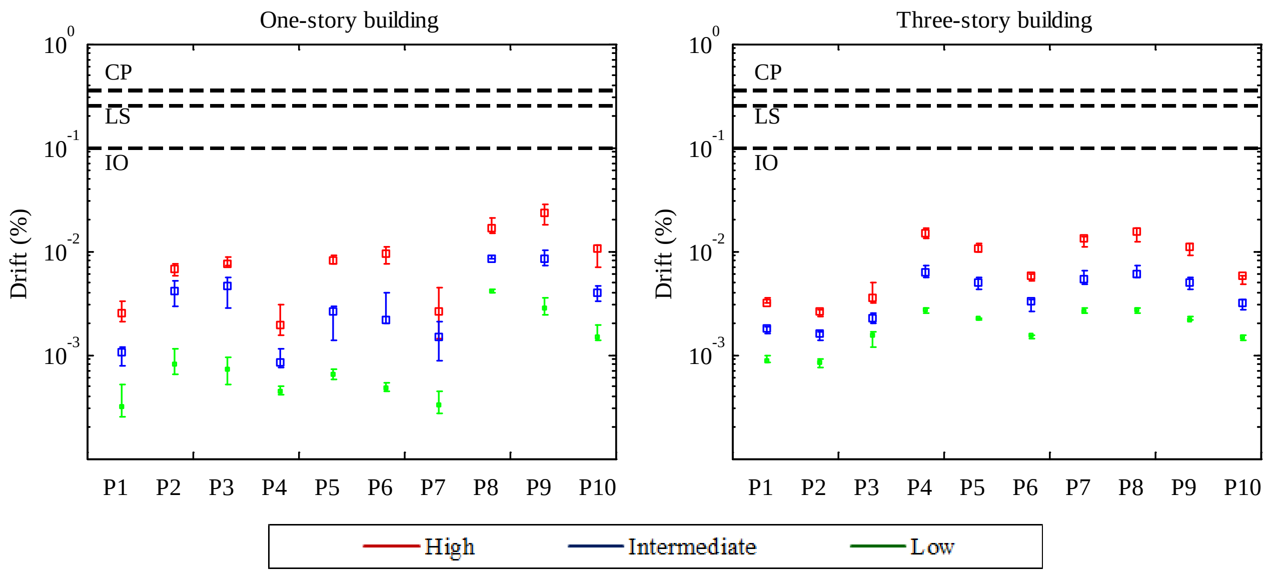


Fig. 5 –Drift demands on the one– and–three–story buildings

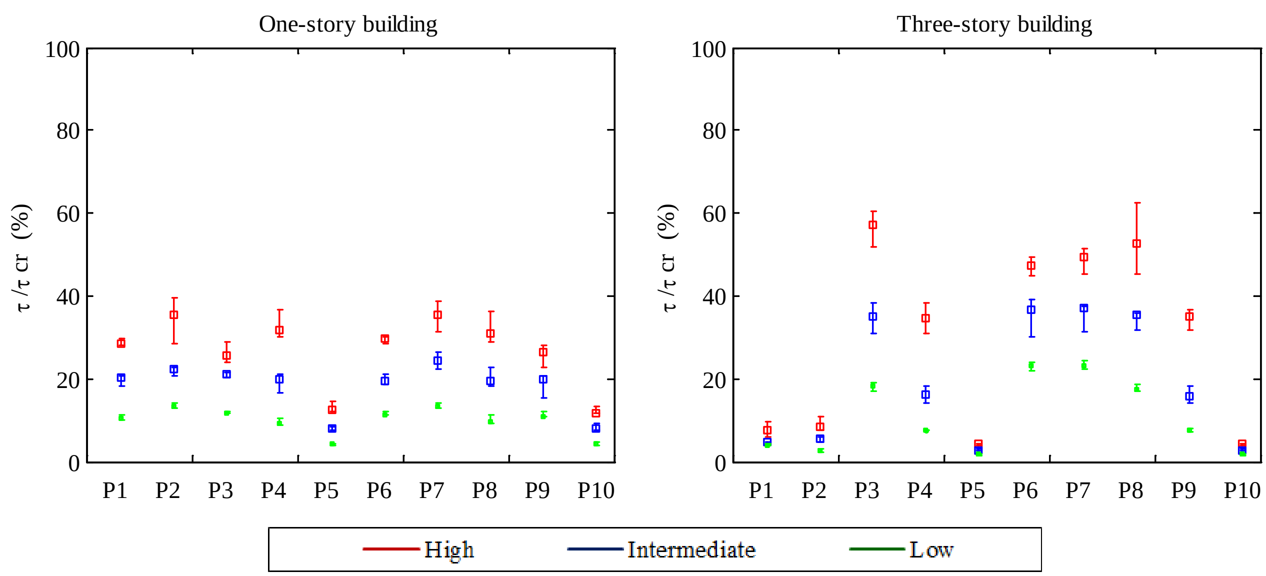


Fig. 6 – Shear stresses demands on one– and–three–story buildings

3. Experimental program

3.1. Materials

Normal–weight concrete having a specified nominal compressive strength f'_c of 21 MPa was selected for the experimental program. To simulate the typical conditions of concrete specification used for industrialized systems in Colombia, accelerating admixtures was added for achieving specified strength at seven days.

Welded wire meshes having 6.0 mm wire diameter, 150 mm spacing on both directions, and minimum yield strength of 485 MPa, were used as web shear reinforcement for the conventional RC practice wall specimen. Deformed rebars used as flexural reinforcement for FRC wall specimens, as well as sliding reinforcement dowels for all specimens, had nominal yield strength of 420 MPa, whereas No. 2 ties had nominal yield strength of 240 MPa.

As part of the research project, an experimental characterization of fiber–reinforced concrete for different dosages and types of fibers was conducted by testing standard (150×150×600 mm) concrete beams (without longitudinal rebars) subjected to bending as specified by ASTM C1609 [8]. Such characterization helped to define different types of FRC mixtures and considered steel and synthetic fibers, and a combination of both (hybrid), two beam specimens were included for each mixture. Fiber properties are listed in Table 1, while the specified values of different FRC mixture properties and average results are reported in Table 2. Behavior of FRC mixtures was evaluated through flexural tests as per ASTM C1609 [8] where the residual strengths at specified net deflections (i.e. L/300, L/150) are identified; then, such values are compared with minimum values prescribed by building codes such ACI 318–08 [5] or NSR–10 [4]. These codes specify minimum residual strength of 90% and 75% at net deflections of L/300 and L/150, respectively. In Table 2, D_f is the fiber content, V_f the concrete volume fraction, f_p is the first peak of flexural strength, f_{max} is the maximum flexural strength, $f_{L/300}$ is the residual strength at a net deflection of L/300, and $f_{L/150}$ is the residual strength at a net deflection of L/150.

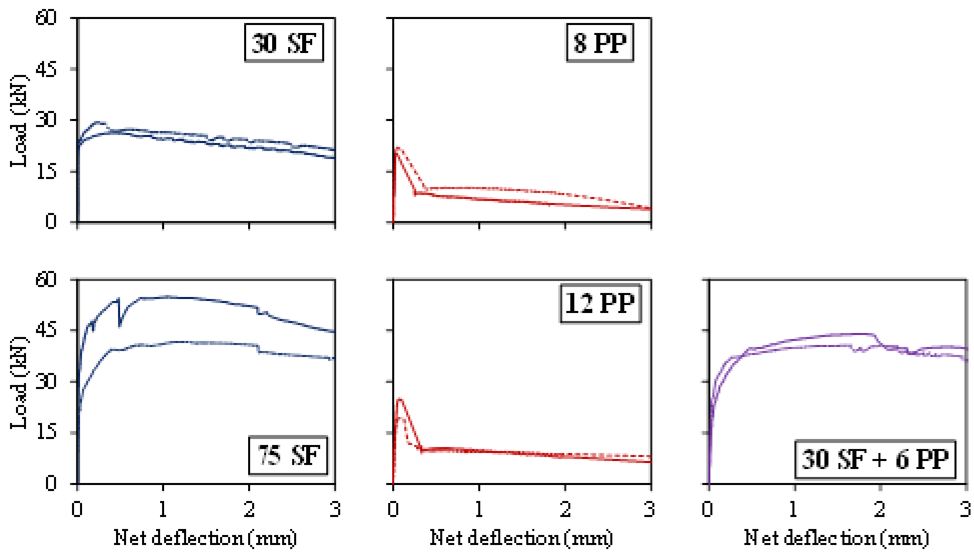
Table 1 – Fiber properties

Type of fiber	Length, l_f (mm)	Diameter, d_f (mm)	l_f/d_f	Tensile strength, f_{ft} (MPa)	Young modulus, E_f (MPa)	Density, γ_f (kg/m^3)
Steel (SF)	60	0.75	80	1225	210000	7850
Synthetic (PP)	55	0.45	122	335	4800	1270

Table 2 – FRC mixture properties and results of flexural tests

Type of fiber	Steel (SF)		Synthetic (PP)		Hybrid (SF + PP)
D_f (kg/m³)	30	75	8	12	30SF + 6PP
V_f (%)	0.38	0.96	0.63	0.94	0.38SF + 0.47PP
f_p (MPa)	3.31	5.52	2.83	2.95	5.12
f_{max} (MPa)	3.72	6.43	2.83	2.95	5.65
$f_{L/300}$ / f_p	0.98	1.15	0.36	0.40	1.10
$f_{L/150}$ / f_p	0.81	0.99	0.18	0.33	0.95

Fig. 7 shows load–deflection curves for the standard beams tested using different FRC mixtures. Based on observed trends and results in Table 2, synthetic–only mixtures were discarded since they do not comply with the minimum values of residual strengths specified by ACI 318–08 [5]. Load–deflection curves for these mixtures tend to have a rather brittle and sudden failure after the maximum strength is attained, and the residual strength is considerably low when compared to steel and hybrid fiber mixtures. Nevertheless, it is worth noting that with a relatively low dosage of synthetic fibers added to the lowest steel fiber dosage (hybrid mixture: 30 SF + 6PP), the behavior improves considerably, and is even comparable to the highest dosage of steel–only mixture, but at a lower production cost.



(a) (b) (c)

Fig. 7 – Load–deflection curves for FRC mixtures: (a) steel fibers; (b) synthetic fibers; (c) hybrid mixture

3.2. Test specimens

The first phase of full–scale wall testing in the research project considered three types of specimens: a conventional RC wall representing common practice governed by minimum code specifications in Colombia (specimen E00), a FRC wall with low flexural reinforcement in its boundary elements (E01), and a FRC wall with high flexural reinforcement (E02). The distinct flexural reinforcements allowed the observation of the behavior for both minimum recommended flexural reinforcement, and pure shear behavior of FRC. Both FRC walls used a steel fiber dosage of 30 kg/m³. Other proposed dosages will be tested in the second experimental phase of the research project.

All wall specimens were geometrically identical, and consisted of a wall having height of 2.30 m, length of 2.40 m (height–to–length ratio of 0.96) and 100 mm–thick web. Fig. 8 shows a scheme of the specimens' dimensions. Flanges on both ends of the wall were provided for representing the boundary conditions of typical walls in the actual buildings, since completely isolated walls are not always common in this system. Furthermore, these flanges provided out–of–plane support for the walls, thus ensuring adequate in–plane testing conditions. Wide and thickness of flanges were 700 mm and 100 mm, respectively. Walls were cast on a 0.8m×0.5m cross–section foundation beam having length of 3.50 m; the dimensions of such beam were controlled by anchoring requirements of specimens to the reaction floor in the laboratory. Sliding reinforcement dowels and flexural reinforcement were cast as embedded reinforcing in the beam for providing adequate development length. The wall web was cast over the surface of the foundation beam as second–phase concrete. Such practice represented the conventional construction process for the industrialized system, where walls and top slabs are cast monolithically for each story, and placing the following walls on the already–cast slab. For the test specimens, the top slab was replaced by a 0.7m×0.35m cross–section top beam, which was intended for acting as a loading beam to transfer forces from the actuator to the wall. Sliding reinforcement dowels for all specimens at the bottom and top of the wall consisted on No. 3 deformed bars spaced 0.30 m on centers, and were embedded 0.30 m inside the web.

The conventional RC wall was reinforced with a welded wire mesh having wires of 6.0 mm diameter and 150 mm spacing on both directions, for both the web and flanges (Fig. 8). L–shaped No. 3 bars were placed staggered along the height of the wall and connected the web and flange meshes, following typical detailing practice for this system.

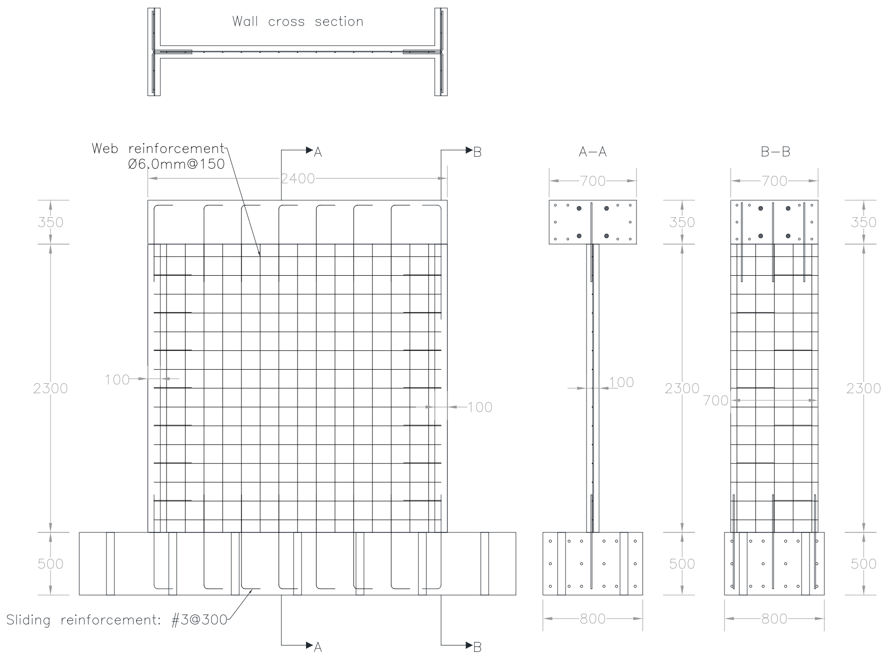


Fig. 8 – Reinforcement layout of the wall specimen representing the conventional practice (dimensions in mm)

Reinforcement layouts for both specimens are shown in Fig. 9 and 10, respectively. As described in the introduction section, flexural reinforcement was provided for FRC wall specimens, due to the absence of welded wire meshes. This reinforcement consisted of longitudinal rebars located in the ends of the wall, at the intersection of the web and flanges, and closed No. 2 ties spaced at 100 mm. In case of the low flexural reinforcement wall (E01), four No. 3 bars were placed on each end, following minimum detailing requirements prescribed by NSR–10 [4] for masonry infilled walls, which may be considered as a similar system. However, this low flexural reinforcement implied that a bending–controlled failure would be achieved before web shear failure, according to expected theoretical capacities. The high flexural reinforcement specimen (E02) was intended to provide the required reinforcement area for attaining the shear capacity of the wall web, thus testing the full loading capacity of the FRC mixture. Therefore, ten No. 4 bars were placed as boundary elements for this specimen. Table 3 summarizes the reinforcement characteristics of each wall specimen, as well as material properties (concrete compressive strength and measured fiber dosage) at the day of testing.

Table 3 – Properties of wall specimens

Specimen	Web reinforcement		Boundary element		f'_c	D_f
	Wire diam. (mm)	Spacing (mm)	Longitudinal bars	Transverse ties	(MPa)	(kg/m³)
E00	6.0	150	–	–	29.7	–
E01	–	–	4 No. 3	No. 2 @ 0.10 m	34.2	31.3
E02	–	–	10 No. 4	2 No. 2 @ 0.10 m	31.5	20.6

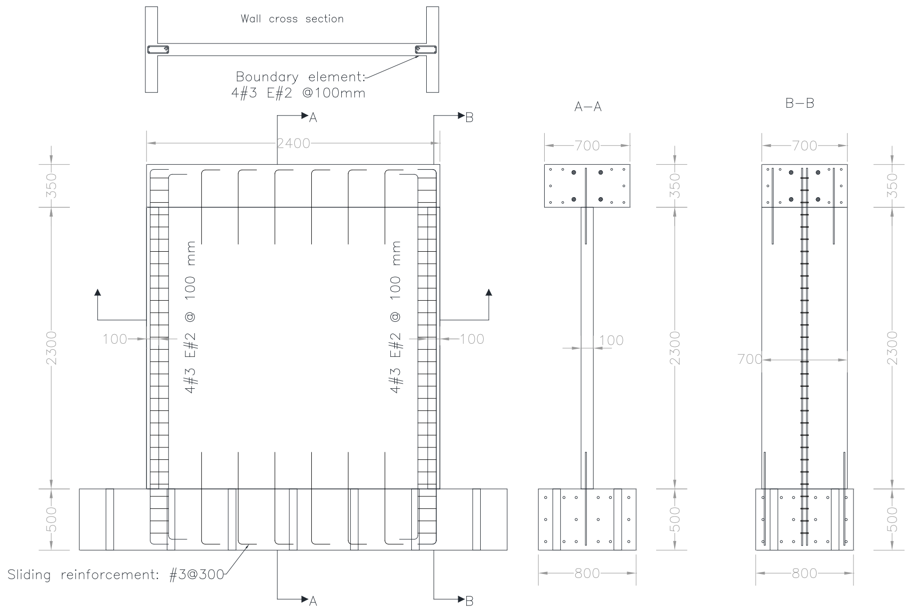


Fig. 9 – Reinforcement layout of the FRC specimen having low flexural reinforcement (dimensions in mm)

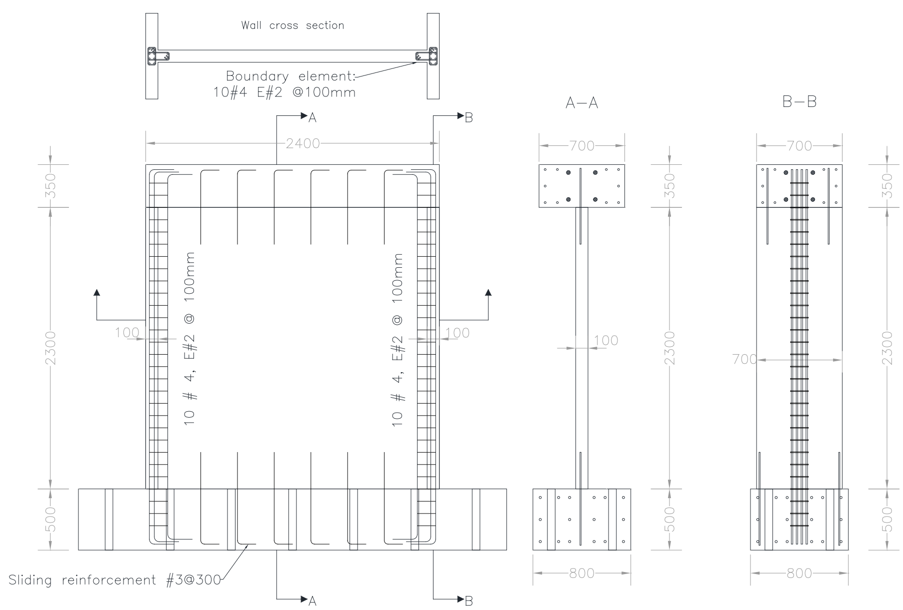


Fig. 10 – Reinforcement layout of the FRC specimen having high flexural reinforcement (dimensions in mm)

3.3. Quasi–static cyclic loading protocol

The loading protocol for testing of walls was based on the provisions of the ACI 374.2R–13 [9] which is proposed for testing of concrete elements under slowly simulated seismic loads. The protocol consisted on increasing amplitude displacements applied to the top beam of the specimen in terms of a displacement control parameter δ. In this case, the theoretical expected cracking displacement was selected as the control parameter. Each phase of the protocol is defined by two cycles of equal amplitude, where the first two phases are defined for 0.5δ and 1.0δ, respectively, and followed by constant increments at each phase of 1.0δ, until the end of the test. For this study, walls were subjected to this protocol until a significant drop of loading capacity was observed (i.e. below 80% of the maximum load).

4. Results and discussion

The failure mode observed for the conventional RC wall test specimen (E00) started as cracking in the bottom of flanges due to bending, and then it extended as a diagonal crack in the lower corner of the web until the load dropped below 80% of the maximum load. Fig. 11 (a) shows the load–displacement hysteretic curve for the wall, and the cracking pattern observed during the test. Strengths predictions corresponding to the lateral load at which flexural capacity would be attained (V_y), the sliding capacity (V_{nfb}), and the shear capacity (V_n) are also presented the hysteresis curve. From these results, theoretical expected capacities are in good agreement of the observed behaviour, since at a lateral load close to the limit associated with the flexural capacity, the wall reached its maximum load, approximately 235 kN. This occurred at a drift of approximately 0.39%.

The FRC wall with low flexural reinforcement experienced a failure associated with sliding at a height of approximately 0.30 m from the foundation beam. This failure started as cracking in the flanges, and continued increasing into the web of the wall as a major horizontal crack moving inwards, until cracks from both ends joined at the center of the web. From this phase, significant sliding displacement between the top and bottom parts of the wall was observed. As shown in Fig. 11 (b), after the initial load drop, a sustained capacity and marked pinching behavior are observed, which is associated to the sliding movement. Although this particular failure mode is not desired, a similar loading capacity to the conventional RC wall was achieved (260 kN); drifts of approximately 0.10% were reached before the maximum load capacity. The height at which the major horizontal crack was observed may suggests the detailing of sliding reinforcement shall be revised.

The behavior under lateral load of the third wall specimen (FRC wall with high flexural reinforcement) was related to pure shear capacity, since major diagonal cracks were observed throughout the test, and the theoretical expected capacity was attained (Fig. 11 (c)). Therefore, important pinching of the hysteresis loops is observed. Furthermore, horizontal drifts of almost 0.5% occurred prior to reaching the maximum lateral load of 600 kN. Nevertheless, a decrease in stiffness due to cracking is observed at low displacement levels (below 0.1%).

According to the results from the nonlinear time history analysis, the three types of walls tested have loading and displacement capacities that satisfy the demands in a typical industrialized concrete wall system. Drift demands from the numerical analysis are below the immediate occupancy performance level (0.1%), and wall specimens displayed a higher displacement capacity than this value. On the other hand, shear stresses acting on walls were below the concrete shear capacity. Experimental results are summarized in Table 4. This comparison validates that design of concrete structural walls in this low–rise system is controlled by minimum provisions established in building codes, and therefore, the absence of reinforcement in the web may be considered technically viable in terms of strength. Furthermore, even low dosages of fiber reinforcement for FRC mixtures and low flexural detailing may provide the necessary lateral load capacity to withstand seismic forces, although the observed failure mode shall be revised under these conditions.

Table 4 – Experimental results of lateral load tests

Specimen	Shear wall capacity			Cracking [+/–]			Maximum Capacity [+/–]		
	V_y (kN)	V_{nfb} (kN)	V_n (kN)	v_{cr} (MPa)	V_{cr} (kN)	Δ_{cr} (%)	v_{max} (MPa)	V_{max} (kN)	Δ_{max} (%)
E00	161.6	228.1	546	0.11	140.0	0.12	0.16	208.9	0.34
				–0.13	–171.4	–0.12	–0.18	–234.5	–0.39
E01	233.8	343.7	578.9	0.15	206.9	0.08	0.19	261.0	0.11
				–0.13	–183.6	–0.076	–0.17	–239.6	–0.10
E02	659	906.6	577.7	0.28	370.5	0.08	0.45	607.9	0.65
				–0.26	–347.3	–0.078	–0.39	–531.8	–0.49

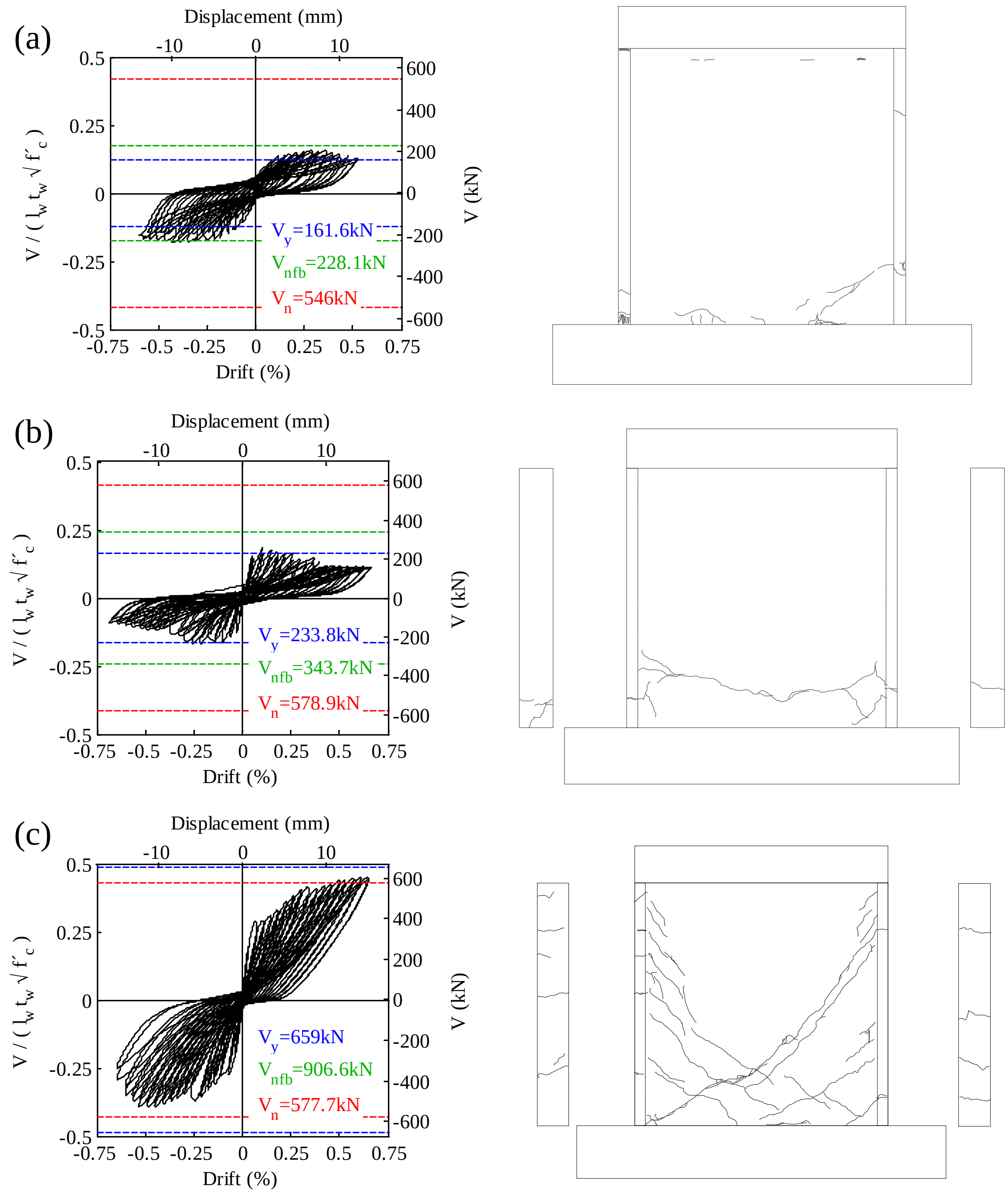


Fig. 11 – Hysteresis curves and cracking patterns of experimental walls: (a) conventional RC wall; (b) low flexural reinforcement FRC wall; (c) high flexural reinforcement FRC wall

5. Final remarks

Based on the preliminary results presented herein, the following conclusions may be drawn:

1. Values of drifts and shear stresses acting on structural walls of industrialized concrete systems are relatively low compared to building code provisions, and therefore, the design of these elements is controlled by minimum values associated with serviceability and durability requirements.

2. Fiber–reinforced concrete having steel fibers, or a mixture of steel and synthetic fibers used in this study comply with performance levels prescribed by the Colombian Building Code NSR–10. Such mixtures may be suitable to be used in structural components of low–rise concrete buildings. Nevertheless, synthetic–only mixtures used in this study do not show an adequate post–peak performance, and therefore were discarded to be used for such walls.
3. Lateral load tests of full–scale walls indicate that the FRC specimen having low flexural reinforcement has a loading capacity comparable to the conventional RC wall, and therefore it may be considered as an alternative reinforcement layout. However, the particular sliding failure mode shall be addressed in further experimental evaluations.
4. The observed behavior under lateral load of all wall specimens can be considered satisfactory, since load and displacement capacities were higher than the expected demands from the nonlinear analysis. To validate these results, different building prototypes shall be analyzed and additional tests on full–scale wall shall be conducted.

6. Acknowledgements

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