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Tensile mechanical properties of the electro-welded wire meshes available in Bogotá, Colombia

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Abstract

Concrete members reinforced with electro-welded cold-drawn wire meshes have experimented deformation capacity lower than that of elements reinforced with conventional ductile reinforcing bars. The tensile mechanical properties of the electro-welded wire meshes available in Bogotá, Colombia, are experimentally assessed in this paper. The mechanical properties evaluated were the yield strength, tensile strength and ductility capacity of the wires in terms of the area reduction, ultimate-to-yield strength ratio, and strain ductility capacity. A statistical analysis allowed contrasting the actual mechanical properties obtained in the experimental campaign, with nominal properties specified by manufacturers and standards. Stress–strain curves of the wires were proposed for code-based design.

Keywords: welded-wire mesh; cold-drawn reinforcement; yield strength; tensile strength; elongation; strain ductility; area reduction; limited deformation; brittle failure.

1 Introduction

Electro-welded wire mesh (WWM) is made of longitudinal and transverse wires welded together at their intersection points. The individual wires that make up the mesh are created by wire-drawing and/or cold rolling of carbon steel rods. The resulting wires may be smooth or deformed. The sizes of the mesh panels and the spacing they exhibit between the wires can be made to the specifications of the given construction project; however, the typical dimensions of commercially available panels in Colombia are 2.35 × 6.00 m. WWMs are mainly used to reinforce slabs and retaining or shear walls. These meshes are widely used in industrial

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construction (e.g., in low-cost housing projects and slabs-on-ground) because they are prefabricated, allowing for rapid placement, reducing the time required and the costs of construction [1, 2]. However, WWMs are also commonly used as web reinforcement for thin walls in medium-rise (up to 16-story) buildings in Latin American countries such a Colombia, Ecuador, Peru, Bolivia, and Mexico [3, 4, 5].

Several investigations have investigated the behavior of reinforced concrete (RC) and masonry elements that use WWM as reinforcement. For example, Kadam *et al*. [6], Barr and Beltrán [7], Quiun *et al*. [8], Yamin *et al*. [9], and Alcocer *et al*. [10] have used WWM for seismic rehabilitation of masonry walls. The results of such studies have demonstrated that the strength of rehabilitated walls wall is equal to or greater than that of the original masonry walls; however, a brittle failure mode may be observed in the rehabilitated walls. Gilbert and Smith [11] studied the behavior of RC slabs with electro-welded wire reinforcement. Those authors also observed that the brittle failure mode of the slabs was controlled by the fracture of wire reinforcement at small deflections. The effects of the ductility of steel reinforcement on the response of RC elements constitute a crucial issue for the performance of earthquake-resistant structures. Riva and Franchi [12], Blandon *et al*. [13], San Bartolomé *et al*. [14, 15, 16], Taira *et al*. [17], and Carrillo and Alcocer [1, 2] reported on the behavior of concrete shear walls with WWM as web shear reinforcement. These studies on RC walls have shown how fragile the behavior of these elements is due to the properties of WWM; thus, limitations regarding their seismic response are described in such studies. For example, Carrillo and Alcocer [1, 2], Blandon *et al*. [13], San Bartolomé [14], San Bartolomé *et al*. [15, 16], and Riva and Franchi [12] show the impact of the wires' low ductility capacity on the consequently low displacement capacity of the walls.

In Colombia, WWM reinforcement is extensively used in the industrialized construction of thin-walled RC buildings. The investigation of the mechanical properties of the WWM that is currently in use by the construction industry at a local level is therefore called for. Although most WWM manufacturers in Colombia offer steel products with quality certification forms, experimental studies that support these claimed properties are lacking. This paper presents and discusses the results of an experimental study that evaluated the main mechanical properties of WWMs with wires having diameters of 4, 5 and 6 mm spaced at 150 mm in the longitudinal and transverse directions. These are the characteristics of mesh commonly used to comply with minimum reinforcement ratios for walls and slabs in Colombia. The meshes studied were manufactured by four of the largest manufacturers of WWM in Bogotá, Colombia. The mechanical properties evaluated were the yield strength (f_y), maximum tensile (ultimate) strength of the wires (f_u), the ductility capacity of the wires in terms of the percentage reduction of area (R_a), the parameter k, defined as the ratio f_u/f_y, and ultimate strain (ε_u). These properties were compared with the nominal parameters required by the ASTM standards and the provisions in the ACI 318-14 [18] Building Code. The parameters for the estimation of the tensile stress–strain curve of the wires and the impact of the mechanical properties of the meshes on the global behavior of RC elements are also presented and discussed in the paper.

2 Mechanical properties of electro-welded meshes in standards

Cold-drawing is a metalworking process for steel in which a thick wire or rod is stretched until it reaches a required diameter [19]. Corrugated or deformed wires are obtained by guiding a steel wire through a series of disks that deform its surface, generating ribs on it. This drawing process causes the wire to suffer an initial plastic deformation that may be greater than the yield strain of the thicker wire rod. This leads to a significantly reduced deformation capacity of the wire, while the yield strength is artificially increased as shown in Fig. 1a. The stress–strain curves for some 5 mm cold-drawn (CD) wires and 9.5 mm low-carbon deformed steel bars tested in the Materials and Structures Laboratory at Nueva Granada Military University (UMNG, Colombia) during the last decade are shown in Fig. 1b. As shown in the figure, the strain capacity of the welded wires is significantly lower than that of the ductile deformed bars. Furthermore, the strain capacity of CD wires tested in Colombia is lower than that of the wires tested in Mexico by Carrillo *et al*. [20] (Fig. 1c).

Some of the mechanical properties of WWM that are relevant for concrete reinforcement can be obtained from a tensile stress–strain curve. The ductility of the wires was estimated in this study using elongation analysis, percentage reduction of area (R_a), ultimate strain (ε_u), and parameter k, defined as the ratio between yield strength and the maximum tensile strength of the wires (f_u/f_y). Although k is defined in terms of strength, it is used by Eurocode EC-2 [21] to normalize the ductility capacity of reinforcement because it partially describes the post-yield response of the wire. For instance, it is widely known that the ductility capacity of the steel product is reduced as the ratio f_u/f_y decreases. On the other hand, larger k values help spread plasticity in the plastic hinge regions of RC members [22]. This is because the equilibrium requirements between adjacent cross sections precludes any localization of strain demand.

The requirements for the elongation and reduction of area tests are defined in each country by technical standards. For example, the ASTM A1064 [23] standard specifies that the required nominal values for f_y and f_u are 485 MPa and 550 MPa, respectively, for Grade 70 (70 ksi, 485 MPa) steel wires used in WWMs for concrete reinforcement. ASTM A1064 requires that the minimum value for the reduction of area for smooth wires is 30%, but it does not specify requirements for elongation tests. Dove [24] argued that the percentage reduction of the area at fracture is a good index of ductility because it measures the amount of deformation that the steel will undergo when subjected to a triaxial state of stress. By contrast, different values for unit elongation can be obtained (at fracture or after fracture), as the these are a function of gage lengths. This is especially true for material that exhibits an elastic, perfectly plastic type of response, as the wires studied herein do. Percentage reduction of in area was also chosen in this study to assess the ductility capacity of the wires, and 30% of R_a was determined as the minimum threshold for accepting a test result.

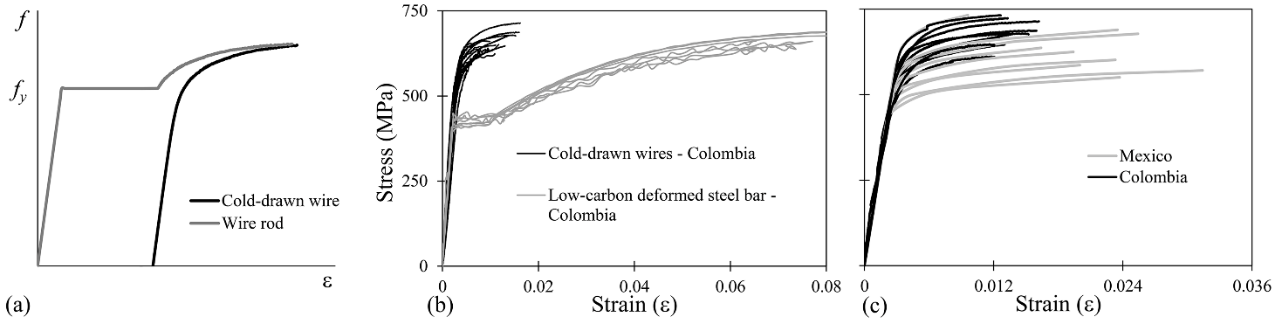

Figure 1. Stress–strain curves: (a) cold-rolled wire rod, (b) typical curves for CD wires and deformed bars, (c) experimental curves of Mexican and Colombian cold-draw wires.

The yield strength (f_y) is the stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain. The two main methods for estimating yield stress are the Extension Under Load (EUL) and the Offset methods [25]. The EUL method is used to accept or reject material that may not exhibit a well-defined disproportionate deformation that characterizes a yield point. Using the EUL method, stress corresponding to a certain level of strain (e.g., 0.5% in Fig. 2) is defined as the yield strength of the wire. In the Offset method, the initial slope of the stress–strain curve is offset by the certain strain (e.g., 0.2% in Fig. 2), and the intersection with the stress–strain curve defines the yield strength. The ASTM A1064 [23] standard states that f_y can be determined using either of the two methods described above. In Fig. 2, the EUL and the Offset methods are depicted in their application to determine f_y from stress–strain curves. In Fig. 2, a hypothetical example is shown in which both methods return the same yield strength in a tensile test on wires. For the circumstance where WWMs are used to reinforce concrete members, ACI 318-14 [18] establishes that f_y can be determined by the Offset method at a strain of 0.2%, or by the EUL method at 0.5% strain if the bar or wire exhibits a well-defined yield point. Although the latter condition does not apply to WWMs, this study used both methods to compare f_y values and evaluate the applicability of the EUL method at 0.5% strain in assessing f_y of electro-welded wires.

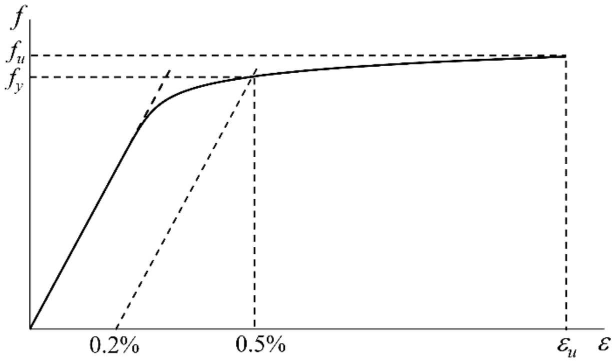

Figure 2. EUL and offset methods for estimating the yield strength of electro-welded wires.

3 Effect of mesh properties on response of concrete elements

Evidence of the limited deformation capacity of CD wires for concrete reinforcement has been reported in the literature over the past three decades. In the United States, Mirza and MacGregor [26] found that the ultimate elongation of smooth and deformed wires tested by Wiss, Janney, and Elstner Associates (WJE) and the Wire Reinforcement Institue (WRI) in 1969 was 22% smaller than the deformation capacity exhibited by conventional reinforcing bars. Dove [24] reported the results of tensile tests in smooth wires with diameters between 2.7 and 10 mm, and deformed wires with diameters between 4 and 10.6 mm. Tests of these two types of wires were carried out under the requirements of ASTM A82 and ASTM A496 available at the time of testing (1983), respectively. The results showed significant differences in the reduction of area values; for example, while the percentage reduction of area for smooth wires varied between 26% and 77%, that of corrugated wires varied between 29% and 46%. In both types of wires, the highest values for the percentages of area reduction were registered for the wires with the smallest diameters.

In Europe, Riva and Franchi [12] evaluated the performance of 18 RC walls with three types of reinforcement: (i) conventional steel bars, (ii) hot-rolled (HR) steel wire mesh, and (iii) CD steel wire mesh. The tensile tests of HR wires showed that the ε_u values were close to 8% and that k was larger than 1.2. The authors found that these values were similar to those of the conventional reinforcement (e.g., $\varepsilon_u > 8$ % and $k > 1.15$), which allowed the HR wires to be classified as reinforcement with adequate ductility for seismic applications, according to the European standard EN 10080 [27]. The CD wires, on the other hand, were classified as low ductility-steel and inappropriate for concrete reinforcement in high-risk seismic zones. Riva and Franchi [12] found in their study that the ductility of concrete walls reinforced with CD wires varied between 60% and 90% of the ductility of concrete walls with HR wires.

In South America, San Bartolome *et al*. [16] studied the behavior of three concrete walls: two walls reinforced with WWM and one wall reinforced with deformed bars. The authors demonstrated the limited ductility of the wire mesh with tensile tests performed on the wires and on the conventional reinforcement. They concluded that walls reinforced with WWM exhibited reduced ductility, which would require the enforcement of a low reduction factor, namely of 3, for the seismic resistance of earthquake-resistant design in Peru. Later, San Bartolome *et al*. [15] evaluated the shear behavior of 12 panels: 6 were reinforced with a high-carbon steel WWM, and 6 were reinforced with ductile steel mesh. The WWM was made of 7 mm diameter wires spaced at 15 and 30 cm, while the ductile mesh had 8 mm diameter bars, spaced at 16.5 and 33 cm. The two types of reinforcement were subjected to tensile tests in which elongations of 2.2% and 15.8% were recorded for the electro-WWM and the ductile steel, respectively. The diagonal compression failure mode or the local failure observed at the load point of the panels hinders the possibility of assessing the direct effects of reinforement ductility on the response of panels. The Peruvian Concrete Building Code [28] only allows WWMs to be used for the distributed reinforcement of so-called "limited-ductility concrete walls" in low-rise structures, up to three stories. For taller buildings, WWMs can be used only in the upper two-thirds of the total height of the building. The maximum story-drift ratio is also limited to 0.5% for buildings with limited-ductility concrete walls [29].

In Oceania, Gilbert and Smith [11] studied the behavior of four simply supported slabs working in one direction, with a span of 3.50 m, and three one-direction slabs with two continuous spans (each of 1.75 m). The slabs were reinforced with CD steel-wire meshes, categorized as class L in the New Zealand standard AS/NZS 4671 [30]. The study included tensile tests of eight samples of wires to determine the yield stress at a strain of 0.2%, tensile strength, and parameters ε_u and k. These tests showed that ε_u was not greater than 3%, and k was less than 1.09. All the simply supported slabs failed in a fragile manner due to rupture of the tension steel in the center of the spans. This occurred for small deflections that varied between 14.5 and 84.1 mm (i.e., l/241 and l/42, where l is the length of the span), when evidence of cracking of the concrete was not visible. Similarly, the continuous slabs failed in a fragile manner, due to the tensile failure of the reinforcement at the positive and negative moment regions, for deflections in the center of the span close to 15 mm (i.e., l/117). Gilbert and Smith [11] concluded that the small plastic deformation capacity of the reinforcing steel was the reason of the fragile response of the slabs, because of concentration of strain demand on a single or a few cracks.

In Mexico, Carrillo y Alcocer [2] analyzed the displacement capacity and shear strength of six walls tested on a shaking table. Three were reinforced with WWMs made of CD steel wire with diameter of 4.11 mm (#6 gauge), and three were reinforced with conventional steel bars. In the tensile tests of the reinforcement, the wires ruptured at the rather small ultimate strain of ε_u = 0.0082. The authors argued that the limited elongation capacity of the reinforcement was the determining factor impacting the displacement capacity of the walls. They also noted that the fragile nature of this type of reinforcement entails that it is inappropriate for use in earthquake-resistant walls. In another study, with the aim of establishing performance levels of squat walls for low-rise housing, Carrillo and Alcocer [1] analyzed 39 cantilever walls subjected to quasi-static tests, and 12 of these were reinforced with electro-WWMs with a diameter of 4.11 mm (#6 gauge). The tensile tests of the reinforcement found a maximum elongation of the wires that varied between 1.4 and 1.9%. The authors showed that there was a dependency between the type of reinforcement in the web of the walls and the level of performance that was achievable. For example, the allowable story-drift ratios for three levels of performance for walls with WWM reinforcement were found to be smaller than those recommended for conventional steel-reinforced walls.

Carrillo *et al.* [20] carried out a complementary experimental study using 360 samples of four types of meshes to evaluate the tensile properties of wires, shear in welded joints, and bending of the meshes produced by five steel mills in Mexico City. They showed that the average value of the reduction of the area of several of the meshes with 4.1 mm wires from one manufacturer did not comply with the minimum requirements set by the Mexican standards. In addition, the values for ε_u and k obtained from the tests did not comply with the requirements of the Eurocode EC-2 [21]. The smallest-diameter wire (3.4 mm) exhibited the most occurrences of lack of compliance in mechanical properties. For instance, welding failure for the smallest wire was observed in 30% of the specimens. In addition, strain ε_u associated with f_u diminished with the reduction in the diameter of the wires. On the other hand, values of f_y as high as 800 MPa were also observed in individual specimens. Carrillo *et al.* [20] observed that the smallest wire gage exhibited lower values of capacity. Those authors argued that the maximum values of the tensile strength of wires should be specified by material standards because substantial increases in strength can be related to reductions in ductility capacity, thus triggering a brittle behavior in a RC member.

4 Experimental program

Three types of meshes commonly used in industrial construction in Colombia were evaluated in this study. The mesh is spaced at 150 × 150 mm, with wire diameters (d_b) of 4, 5 and 6 mm. The material was selected from four steel mills, identified as A, B, C and D. This study evaluated the mechanical properties f_y, f_u, and the ductility capacity of the wires, defined in terms of (i) the percentage reduction of area (R_a), (ii) the ultimate strain (ε_u), and (iii) parameter k. The ASTM A1064 [23] states that the material is strength compliant if the average tensile strength value of three specimens is greater than or equal to the minimum required strength (i.e., 485 MPa for f_y and 550 MPa for f_u) and if none of the three specimens falls below 80% of the minimum strength requirements. ASTM A1064 requires that two additional samples must be tested whenever there is a test specimen that does not comply with capacity requirements. Following these criteria, this study established that four tests would be conducted per wire diameter; hence, a total of 48 tensile tests were developed (4 tests/diameter × 3 diameters × 4 mesh manufacturers). As per the discussion presented above, a 30% area reduction was used as the minimum value required to fulfill ductility requirements.

4.1 Mesh type and size definition

ACI 318-14 Building Code [18] and NSR-10 Colombian code for earthquake-resistant construction [31] contain provisions for the use of wires in RC members and minimum reinforcement quantities. For example, sections 20.2.1.7 of ACI 318-14 and C.3.5.3.5 of NSR-10 preclude the use of deformed wires with diameters smaller than 5.6 mm or greater than 16 mm to reinforce concrete. Sections 11.6.1 of ACI 318-14 and C.14.3 of NSR-10 establish that the minimum steel ratio for vertical (ρ_v) and horizontal (ρ_h) reinforcement in shear walls must be at least 0.12% and 0.20%, respectively, when deformed or smooth electro-welded wire reinforcement is used. Regarding seismic provisions, sections 18.10.2 of ACI 318-14 and C.21.9 of NSR-10 indicate that the minimum-reinforcement ratio distributed in the web (ρ_{min}) must be 0.25% for special structural walls. Eq. (1) defines the area of a single vertical or horizontal wire (A_s) in terms of the wall reinforcement ratio (ρ), wall thickness (t), and wire spacing s.

$$A_s = \rho \times t \times s \tag{1}$$

Table 1 shows the minimum diameters of wire required to fulfill the aforementioned minimum code-based provisions for walls. Accepting the typical wire spacing as s = 150 mm, diameters shown are for various wall thicknesses. Although 80 mm may seem rather thin, this thickness is widely used in Colombia for the construction of low-rise buildings. Meshes with wire diameters of 5 and 6 mm as vertical and horizontal reinforcement, respectively, may comply with the minimum reinforcement ratios shown in Table 1. However, ACI 318-14 does limit the use of deformed wire as reinforcement in structural systems with seismic detailing. Consequently, the equivalent minimum d_b values that are shown in the third row of Table 1 apply only to the Colombian case. Table 1 also shows the minimum wire diameters required to fulfill the ACI 318-14 and NSR-10 minimum-reinforcement provisions for slabs of various thicknesses (h). The reinforcement areas shown (A_s-values) are given per unit-length of slab (i.e., per each 1 m of slab). As shown in the table, wire diameters of 4, 5, and 6 mm can be used to comply with minimum reinforcement requirements for shrinkage and temperature (e.g., 0.16%) in slabs with thicknesses of 80, 100 and 120 mm, respectively.

Table 1. Minimum wire diameter to comply with minimum reinforcement requirements of different thicknesses of walls and slabs

Element	Direction	ρ	Wall or slab thickness, t (mm)					
			120		100		80	
			A_s (mm^2)	d_b (mm)	A_s (mm^2)	d_b (mm)	A_s (mm^2)	d_b (mm)
Walls	V	0.12%	21.6	5.2	18.0	4.8	14.4	4.3
	H	0.20%	36.0	6.8	30.0	6.2	24.0	5.5
	V and H	0.25%*	45.0	7.6	37.5	6.9	30.0	6.2
Slabs	---	0.16%	1.40	5.2	1.09	4.6	0.78	3.9

V = vertical, H = horizontal, *only applies for the Colombian case, where deformed wires are used for seismic applications.

4.2 Samples and test setup

For yield and ultimate strength tests, 48 specimens were obtained, matching each type of mesh for each of four manufacturers. Each specimen was 600 mm long and included three welded intersections at the center of the wire to be tested because ASTM A1064 [23] specifies that no fewer than 50% of tests shall be carried out across welds. The transverse wire forming the welded intersection was also extended approximately 25 mm beyond each side of the intersection. Additionally, post-yield strain gages were bonded to the surface of the bars as a second source of strain information (Fig. 3a). An MTS extensometer, model 632.54F-20, was also mounted on the test specimens, which were tested on a MTS loading equipment rated as having a capacity of 100 kN (Fig. 3b). The test speed was set to be near to the maximum value of 690 MPa/min, in compliance with ASTM A370 [25].

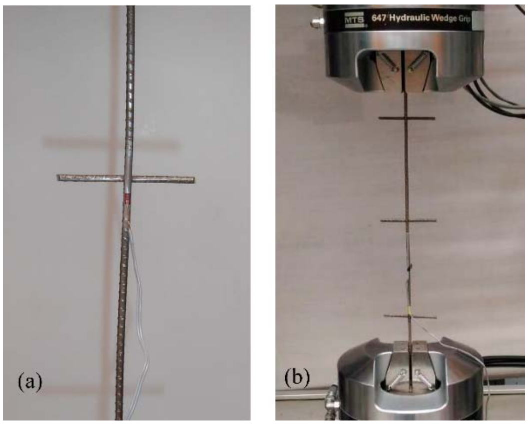

Figure 3. Test setup and instrumentation: (a) strain-gauge on sample, (b) loading equipment.

5 Results and discussion

The results of the tensile tests are presented and discussed in this section. The statistical analyses used such summary statistics as the mean (X), the coefficient of variation (CV) and the 2nd percentile ($P2$). These parameters were estimated using Eqs. (2) to (4).

$$X = \frac{1}{N}\sum_{i=1}^{N} x_i \tag{2}$$

$$CV(\%) = \frac{S}{X} \times 100\% \tag{3}$$

$$P2 = -2.05S + X \tag{4}$$

where x_i is a random variable that represents the response of the i^{th} sample, N is the sample size and S is the standard deviation of the sample. When CV was less than 10%, the variation in the results was considered low, it was considered acceptable for CV values between 10% and 30%, and it was considered large for CV values greater than 30%. The $P2$ value, above which 98% of the observations may be found, is accounted a conservative measure of performance because there is only a 2% probability that the mechanical property is lower than the value found. This allows a minimum nominal value of the mechanical property under study to be defined, as established in the Mexican standard NTC-C [32] for the design of concrete structures. To evaluate R_a, the ASTM A370 [25] standard states that the diameter of the reduced section must be obtained by joining the ends of the fractured specimen and measuring the dimensions of the smallest cross section, to an approximation of 0.025 mm. Parameter R_a of the wires was determined using Eq. (5).

$$R_a = \frac{D_i - D_f}{D_f} \times 100\% \tag{5}$$

where D_i is the nominal diameter of the wire and D_f is the average diameter, measured in the fracture section of the wire.

5.1 Tensile test of the wires

Table 2 shows the measured values of f_y associated with the strains specified by the EUL and the Offset methods prescribed by ASTM A1064 [23]. Table 2 also indicates compliance with the requirements for each type of mesh and each manufacturer. The results show that mean value of f_y associated with a strain of 0.5% in EUL (621 MPa) is roughly equal to the value of 0.2% found by in Offset (629 MPa). Actual yield strengths were in the range of 506 and 713 MPa, and hence, all specimens complied with the minimum requirements for f_y, as specified by ASTM A1064. Furthermore, the average yield strength of individual specimens at 0.5% for EUL and 0.2% for Offset were at least 28% (621/485 MPa) and 30% (629/485 MPa) larger than the minimum of 485 MPa required by ASTM A1064. It is noted that, on average, with increasing wire diameter, f_y increases while *CV* decreases. This can be confirmed by the results of the specimens of manufacturer B, which exhibited the smallest *CV* as well as the smallest yield strength. The specimens of manufacturer D exhibited, on average, the largest *CV*, while those of manufacturer A had, on average, the largest yield strength. Table 2 also shows the mean values and the *CV* and *P2* of the measured values of the tensile strength (f_u) of the wires. In the table, compliance with ASTM A1064 is also given. The measured f_u values for the wires were, on average, 25% (685/550 MPa) greater than the minimum of 550 MPa that is required by ASTM A1064.

Table 2. Yield and tensile strengths of wires in electro-welded meshes

d_b (mm)	Manufacturer	Yield strength, f_y				Tensile strength, f_u	
		0.50% EUL		0.20% Offset			
		X, MPa	*CV*, %	*X*, MPa	*CV*, %	*X*, MPa	*CV*, %
4	A	683	5.1	713	3.5	767	1.6
	B	542	12.3	558	6.0	615	5.3
	C	570	7.5	571	8.2	628	8.6
	D	634	6.5	638	6.2	728	4.7
5	A	617	6.1	630	3.4	671	1.3
	B	598	2.8	597	1.2	689	1.5
	C	604	5.0	610	4.6	670	1.2
	D	619	4.3	620	5.2	691	2.7
6	A	635	1.5	665	1.6	717	2.1
	B	559	1.4	559	2.0	608	3.2
	C	684	1.4	691	1.3	715	3.6
	D	678	5.5	686	5.0	724	7.6
X	For all manufacturers	621		629		687	
S		54.2		55.9		53.8	
CV, %		8.7		8.9		7.8	
P2		510		514		577	

A graphical representation of the distribution of data for measured strength is given in Fig. 4, which shows the histograms of yield and ultimate strength constructed to indicate the results for all 48 specimens. The frequency of the observed values of f_y and f_u for all combinations of wire diameters and manufacturers is depicted in the figure. A normal distribution (dotted line) is fitted on the histograms using the means and standard deviations reported in Table 2. The *P2* values for f_y and f_u are also shown in Fig. 4.

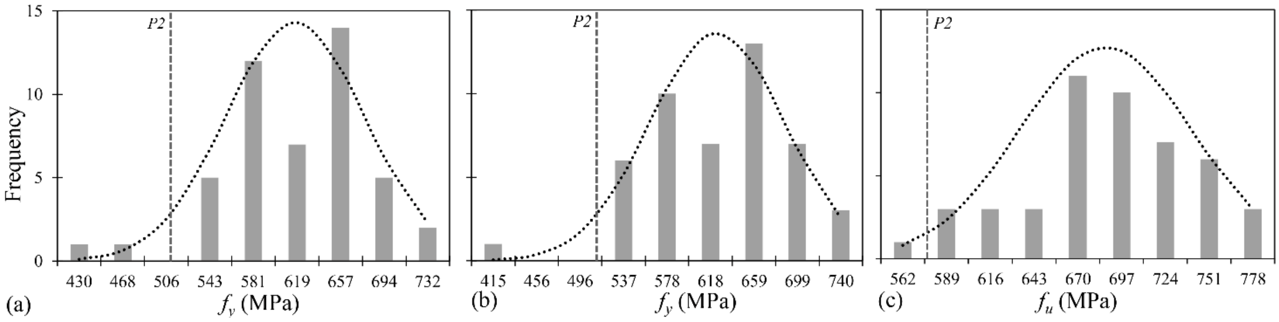

Figure 4. Frequency distributions: (a) f_y per EUL to 0.5%, (b) f_y per Offset to 0.2%, (c) tensile strength.

Table 3 shows the average values, sorted by manufacturer and wire diameter, of parameters *k*, ε_u, and R_a, together with their corresponding coefficients of variation. With the exception of the 6 mm wires made by manufacturer C, the *k*-values obtained from f_y at the 0.2% offset are greater than the minimum values reported by Riva and Franchi (2004) for CD wires (i.e., k = 1.05). On average, the *k* values obtained from f_y at 0.5% EUL and 0.2% offset are also similar (e.g., 1.11 and 1.10, respectively). Sections 20.2.2.5 of ACI 318-14 [18] and C.21.1.5 of NSR-10 [31] establish a minimum *k* value of 1.25 for the steel reinforcement of structures with special energy dissipation (SED) capacity. Although a *k* value is not established when WWMs are used to reinforce concrete, ACI 318-14 restricts the use of WWM in structures with SED capacity.

Table 3. Ductility parameters of the wires in electro-welded meshes

d_b (mm)	Manufacturer	$k = f_u/f_y$		ε_u, %		R_a	
		0.5% EUL	0.2% Offset	*X*	*CV*, %	*X*	*CV*, %
4	A	1.12	1.08	1.27	36.4	28.0	6.6
	B	1.13	1.10	1.17	4.7	28.8	9.0
	C	1.10	1.10	1.09	73.2	30.8	6.7
	D	1.15	1.14	1.43	43.3	27.4	6.4
5	A	1.09	1.07	0.85	12.0	26.4	8
	B	1.15	1.15	1.09	6.9	25.4	9
	C	1.10	1.09	1.11	30.3	27.1	9
	D	1.12	1.12	1.46	9.8	28.7	11
6	A	1.10	1.08	1.33	26.5	20.6	8
	B	1.09	1.09	0.96	44.7	31.6	12
	C	1.05	1.03	0.82	39.7	23.7	12
	D	1.11	1.09	0.87	15.5	24.4	6
X	For all manufacturers	1.11	1.10	1.11		27.0	
S		0.051	0.044	0.40		3.70	
CV, %		4.6	4.0	36.1		13.7	

The average values of ε_u in Table 3 for the tensile strains of the wires tested varied between 0.82% and 1.46%. This means that none of the wires tested in this study reached an ultimate strain greater than that reported by Riva and Franchi [12] for 12 mm CD wires (i.e., ε_u = 3%), nor that reported by Gilbert and Sakka [33] for 6, 7.5 and 9 mm CD wires (i.e., ε_u = 2.4%). It is worth recalling that the Gilbert and Sakka tests were categorized as low ductility by the AS/NZS 4671 [30] standard. The average value of ε_u for all manufacturers in Table 3 was 1.11%. This low value is approximately equivalent to 44% of the minimum value established by Eurocode EC-2 [21] for low-ductility reinforcing steel. In addition, the coefficient of variations as high as 73% revealed significant and undesirable differences of values of ε_u between different diameters and manufacturers of wires.

The minimum acceptance value for R_a of 30% established by ASTM A1064 [23] was satisfied only by manufacturers B and C for wires with diameters of 6 mm and 4 mm, respectively. However, the reported values of R_a are not significantly greater than the minimum value. The *CV* values for R_a are considered acceptable because they varied between 6% and 12%. As discussed in Section 2, ASTM A1064 establishes a minimum value of R_a for smooth wires but does not establish one for corrugated or deformed wires. This lack can be attributed to the fact that there is no clear methodology by which the diameter and the corresponding area in the failure section of the corrugated wires can be calculated with good precision. The presence of ribs in the failure zone makes it cumbersome to measure the diameter of the wire. Consequently, the methodology used here to determine the reduced diameter at failure and the corresponding percentage reduction of the area of corrugated wires, in a way similar to plane wires, may be inadequate.

In Table 4 is shown a summary of the statistical analyses of this study, including the mean, *CV*, minimum (*Min*) and maximum (*Max*) values of the measured data for f_y (associated with the 0.5% EUL and 0.2% Offset strains), f_u, ε_u, and R_a of wires with diameters of 4 mm, 5 mm, and 6 mm. In Table 4, the *P2*-values of f_y and f_u are also shown. It should be noted that the *P2* values of f_y at 0.5% EUL and f_y at 0.2% Offset for wires with diameters of 5 mm and 6 mm were greater than the minimum value (485 MPa) prescribed by ASTM A1064 [23]. However, this was not true for the 4 mm diameter wires, whose *P2* values for f_y and f_u are below 98% of the minimum requirements, per ASTM A1064. In terms of the ductility parameters, mean values for ε_u and R_a decreased as the wire diameters increased. Although a similar trend had already been reported by Dove [24], Carrillo *et al*. [20] observed a contrasting trend, that is, they found that the ductility capacity of wires was reduced as the wire diameter decreased. It can also be observed in Table 4 that the values of *CV* for ε_u and R_a are high, especially for parameter ε_u, where *CV* varied between 33% and 43%.

5.2 Proposed stress–strain curves

The stress–strain curves recorded in this study for each wire diameter for each manufacturer are shown in Fig. 5. Mirza and Mac-Gregor [26] proposed a modified model of the Ramberg–Osgood curve to allow the calibration of a stress–strain curve of electro-welded wire from empirical data. The proposed model is defined by Eq. (6).

$$\varepsilon = \frac{f}{E_s} + \left(\varepsilon_u - \frac{f}{E_s}\right)\left(\frac{f}{f_u}\right)^{20} \tag{6}$$

where ε is the strain, f is the corresponding tensile stress in MPa, f_u is the maximum tensile strength in MPa, ε_u is the ultimate strain or the strain corresponding to f_u, and E_s is the modulus of elasticity of the steel. In this study, a theoretical value of 200,000 MPa was adopted for E_s. The stress–strain curves were computed using Eq. (6), and the mechanical properties measured in this study given as the model curves in Fig. 5. Those curves were calculated using the *P2*-values of f_u and the mean values of ε_u found in Table 4. Mean values of ε_u instead of *P2* were used in Eq. (6) because no minimum value for ε_u is prescribed by either ASTM A1064 [23], ACI 318 [18] or NSR-10 [31].

Table 4. Summary of the main mechanical properties of wires of WWMs.

Mechanical property	Method	Statistical parameter	d_b (mm) 4	5	6
Yield strength, f_y (MPa)	EUL (0.5%)	*X*	612	610	641
		Min	467	563	551
		Max	732	652	719
		CV, %	11.3	4.4	8.6
		P2	469	554	528
	Offset (0.2%)	*X*	624	614	648
		Min	525	582	549
		Max	740	658	722
		CV,%	11.5	4.0	9.1
		P2	477	564	527
Tensile strength, f_u (MPa)		*X*	689	681	691
		Min	562	658	587
		Max	778	714	763
		P2	538*	650*	573*
Ultimate strain, ε_u (%)		*X*	1.24*	1.13*	0.95*
		Min	0.51	0.74	0.56
		Max	2.26	1.62	1.58
		CV,%	42.7	25.0	33.2
Reduction of area, R_a		*X*	29.0	26.9	25.2
		Min	25.3	23.1	18.6
		Max	33.1	31.9	34.8
		CV,%	7.7	21.6	19.2

* values used to construct the "Model" curves.

The results of Fig. 5 confirmed that the functional form of the model proposed by Mirza and MacGregor [26] is a suitable representation of the actual performance of the wires produced in Bogotá, Colombia, that were examined in this study. It is worth noting that the curve that corresponds to wires with diameter of 5 mm (Fig. 5b) fits any of the experimental curves reasonably well. On the other hand, the curves representing wires with diameters of 4 mm (Fig. 5a) and 6 mm (Fig. 5c) indicate a conservative lower bound for the experimental response because of the larger variation of the measured results. This is supported by the large values of *CV* associated with f_u for wires of these diameters, as opposed to 5 mm diameter wires (see Table 4).

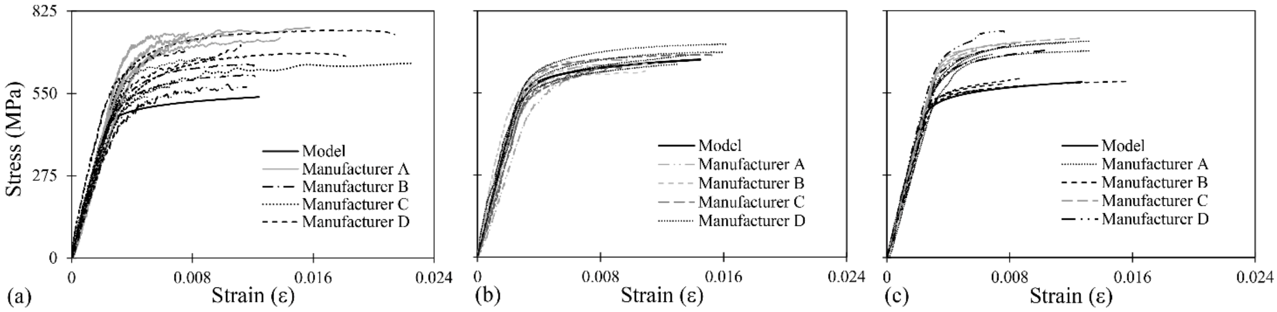

Figure 5. Measures and computed strain-stress curves for the three wire diameters: (a) 4 mm, (b) 5 mm, (c) 6 mm.

6 Impact of the wire-mesh reinforcement material behavior on the mechanics of RC members

Wires exhibiting an elastic–perfectly plastic (EPP) type of response with limited plastic deformation capacity will greatly influence the behavior of RC members undergoing large deformations. First, the lack of strain hardening (SH) under flexural tension action may trigger the localization of plastic demand on a single crack after the onset of yielding [34]. Second, the limited rupture strain of reinforcing steel may negate its compliance with the assumption in classical design of under-reinforced flexural elements, which states that in extreme flexural compression, fiber reaches the ultimate strain of ε_{cu} = 0.003, while the extreme steel fiber on the opposite site of the neutral axis yields but does not rupture.

For the first case, Fig. 6 shows a prismatic beam under uniform moment demand. The tension chord is idealized as a set of springs in series, and the equilibrium equations impose on them of the constraint of carrying the same force. The central spring (CS) is assumed to have a slightly lower yield strength as the yielding of the tension chord begins in that critical section before moving to the adjacent chord ones (e.g., before to the left hand side and right hand side springs, denoted as LS and RS in Fig. 6, respectively). The two stages of the moment-rotation response shown are (a) the onset of the CS yielding and (b) the incremental rotation beyond the yielding. Two different cases of a bilinear constitutive relationship of the reinforcement were examined: one was an EPP relationship (case b.1), and the other was an SH relationship (case b.2). In case b.1, after the onset of yielding, the CS elongates and reaches strain $\varepsilon_{s,b1\mathrm{CS}}$ while maintaining the same stress f_y, due to the lack of SH. The left and right springs maintain the same axial force; hence, the strain demand on them is unchanged. This translates into a single crack opening at the CS location, where all plastic demand is concentrated, resulting in nonductile global moment-rotation behavior. In case b.2, after the onset of yielding, the CS elongates to strain $\varepsilon_{s,b2\mathrm{CS}}$ while it also increases its stress to $f_{s,b2}$, thanks to its SH nature. To complying with equilibrium, the stress demands on LS and RS must both increase to $f_{s,b2}$ as well and hence require an elongation to strain $\varepsilon_{s,b2\ \mathrm{LS/RS}}$. Because of compatibility rules, the result is a uniform distribution of crack openings, resulting in ductile flexural behavior mimicking the constitutive stress–strain curve. In this case, plasticity spreads, while for case b.1, plasticity localizes. In Colombia, the failure mode of the thin concrete wall specimens reinforced in the web using CD WWM was characterized by ruptures in the steel, mainly as a result of the concentration of plasticity to a single crack, located at the interface between the wall and the foundation [13].

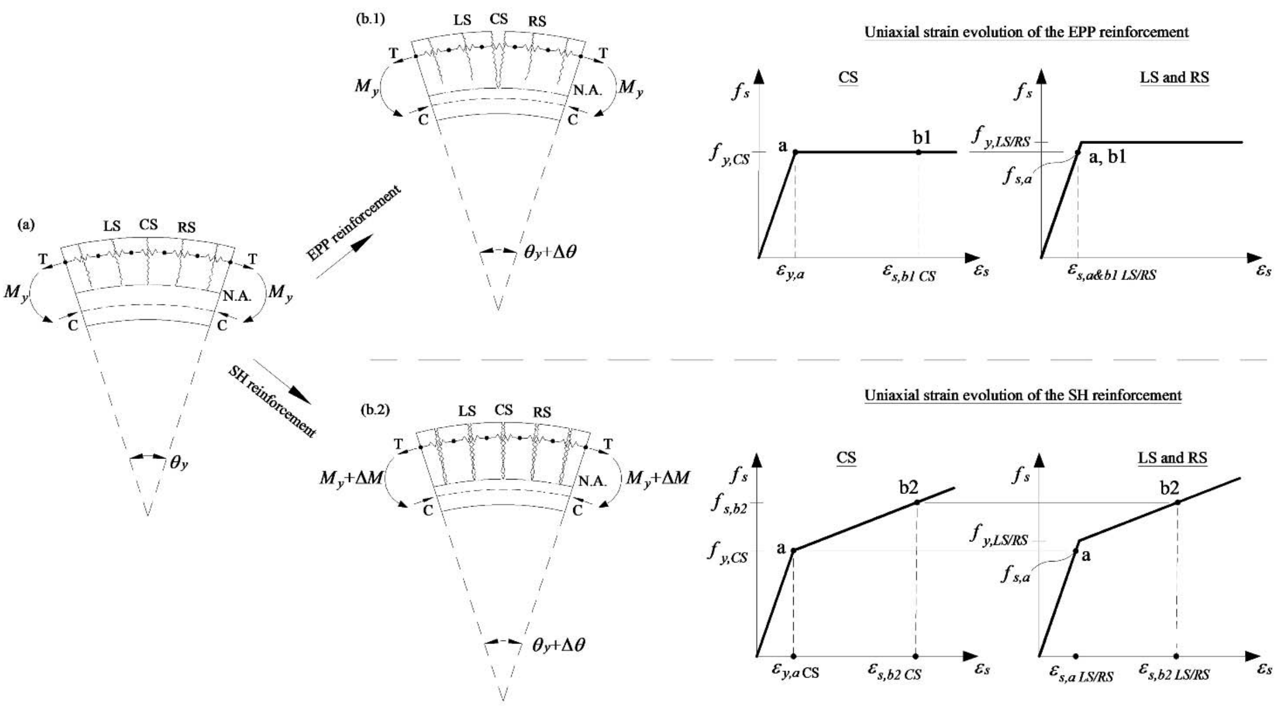

Figure 6. Spread of plasticity evolution for a beam element under uniform moment for two different reinforcing steel behaviors.

The limited rupture strain of the CD wires poses its own problems. Section 22.2 of ACI-318-14 [18] defines maximum strain for extreme concrete compression fiber as $\varepsilon_{cu} = 0.003$, which is supported by empirical data in Mattock *et al.* [35] and in Kaar *et al.* [36]. According to sections 22.3 and 22.4 of ACI-318-14, this assumed strain limit should be met for the estimation of nominal flexural strength or the combined flexural and axial strength of the RC members, respectively. The question arises whether this assumption holds, given the limited rupture capacity of the wires studied here. In Fig. 7a, a T-shaped RC cross section is shown that is representative of the critical section of a thin shear wall in a multistory building. The web and flange thicknesses are $t_w = t_f = 120$ mm. The other dimensions of the cross section are the web length l_w and the flange length l_f. The section is uniformly reinforced with a wire mesh on the web and a flange with longitudinal reinforcement ratio ρ. The nominal moment M_n is estimated for the direction in which the flange is compressed, under various levels of axial load $P = A_g f'_c$. The selected reinforcement model for analysis is depicted in Fig. 7b, which is representative of the experimental curves shown in Fig. 5.

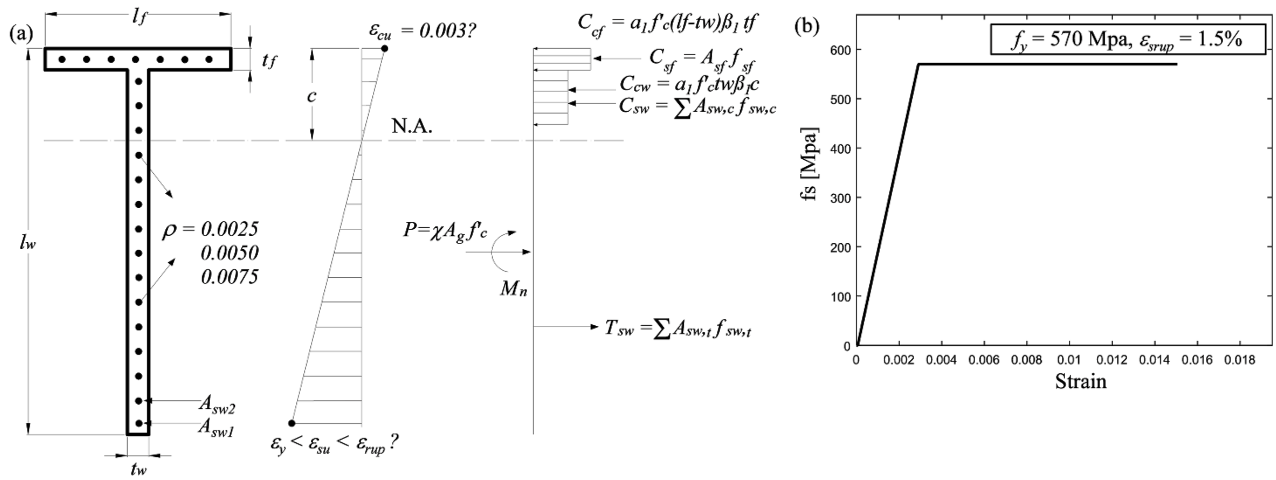

Figure 7. Section analysis variables: (a) T-shaped section geometry, hypothetical normal strain distributions, and external and internal force for estimation of nominal strength; (b) constitutive model used for the reinforcement.

A numerical experiment was developed to sample all plausible pairings of web and flange lengths in the ranges $1.0 < l_w < 9.0$ m and $1.0 < l_f < 6.0$ m, which is representative of the wire used in thin-wall construction in Colombia [5]. For a given set of ρ and P, 900 pairs [l_w, l_f] were analyzed to estimate M_n, evaluating whether the extreme fiber in compression reaches its limiting value of 0.003 prior to rupturing the most extreme steel fiber on the edge of the web. This analysis aids the understanding of the situation if the classical hypothesis put forward in the section analysis holds. An example surface relating the maximum compressive strain $\varepsilon_{cu} \leq 0.003$ with values of l_w and l_f is shown Fig. 8a, as recorded in each of the 900 section analyses. Pairs [l_w,l_f] on the plateau comply with the design hypothesis, and they are bounded by the straight line of Eq. (7).

$$l_f \leq b + \mathrm{m}\, l_w \tag{7}$$

where coefficients b and m are a function of ρ and P (see Table 5). In Fig. 8b, example relations of ε_{cu}, l_w, and l_f are shown for four sets of ρ and P values. It was observed that for minimum reinforcing ratio $\rho = 0.0025$ with low axial load, no cross section complied with the design hypothesis. The addition of reinforcing steel varied the curve marginally, with only rectangular cross sections reaching ε_{cu} = 0.003 at the extreme fiber in compression. The addition of larger compressive forces affected the demand on the compression side more, which resulted in an apparent expansion of the compliance plateau. It was noted that addition of a flange to an otherwise rectangular wall greatly influenced the demand for deformation on the compression side. This is because the addition of large area in the compression block allowed equilibrium to be reached with a small neutral depth *c* of the axis, which in turn exacerbated the demand on the steel on the tension side.

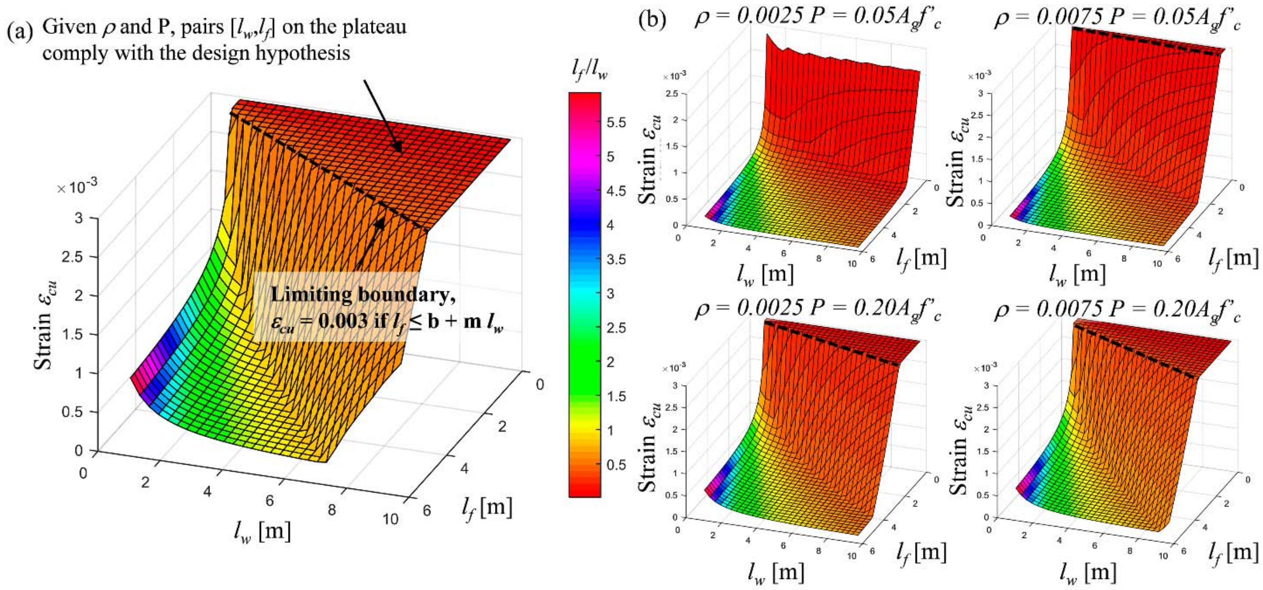

Figure 8. Maximum strain in compression versus web and flange length: (a) figure interpretation for a given ρ and *P* value; (b) ultimate compressive strain curves for several values of ρ and *P*.

Table 5. Parameters *m* and *b*

ρ	χ %	*m*	*b*
0.0025	5	0.000	0.12
0.0025	10	0.023	0.12
0.0025	20	0.184	0.32
0.0025	30	0.460	0.52
0.0050	5	0.000	0.12
0.0050	10	0.069	0.12
0.0050	20	0.253	0.32
0.0050	30	0.506	0.72
0.0075	5	0.046	0.12
0.0075	10	0.138	0.12
0.0075	20	0.299	0.52
0.0075	30	0.598	0.72

In terms of the displacement capacity, RC buildings designed according to NSR-10 Colombian code for earthquake-resistant construction [31] are expected to deform up to a maximum lateral drift of 1.43% for the design earthquake, which has a return period of 475 years, without collapsing and with limited structural damage. If the ultimate strain capacity of steel wires used for the reinforcement of concrete walls is considerably lower than that of the minimum value mandated by code, a design drift limit significantly lower than 1.43% should be used for earthquake-resistant design for concrete-wall buildings. The results of a shake table [2] and quasi-static cyclic tests [4, 13] have confirmed that displacement capacity of concrete walls reinforced in the web using CD steel-wire meshes should be close to the story-drift limit of 0.5% prescribed by the Peruvian code for earthquake-resistant buildings [29].

On the other hand, because minimum web steel reinforcement is intended to maintain the inclined diagonal tension cracking load, design codes allow a reduction of the web–steel ratio in proportion to the increase in yield strength relative to that of Grade 60 steel [37]. This effect is considered in ACI 318-14 [18] and in NSR-10 [31]. However, reductions in steel ratio should be implemented when greater-yield reinforcement exhibits a minimum strain capacity relative to the minimum ductile behavior. Due to the sudden fracture of WWM and the corresponding brittle and undesirable failure mode of wall specimens, as reported by Carrillo and Alcocer [2], Quiroz *et al.* [4] and Blandon *et al.* [13], a reduction of the web steel ratio in proportion to the increase of yield strength should not be allowed by seismic design codes [37] for walls with web shear reinforcement made of WWM similar to those reported in this study.

7 Conclusions and recommendations

The results of this study confirmed that wires in the electro-welded wire meshes (WWMs) available in Bogotá, Colombia, complied with the minimum values of ultimate strength (f_u) specified by ASTM A1064 [23]. It was observed that the yield stress (f_y) of the electro-welded wires determined by the Offset method at 0.2% strain and that determined by the EUL method at 0.5% strain were comparable. Although ACI 318-14 [18] allows f_y to be determined by the EUL method at 0.5% strain only if the wire exhibits a well-defined yield point, the results of this study demonstrated that the two methods can be used to determine the f_y of the electro-welded wires studied herein. Although the minimum value of the 2nd percentile (*P2*) is not prescribed by ASTM A1064, the results of the study are evidence that the *P2*-values of f_y and f_u of the wires having diameter of 4 mm are lower than the minimum values for f_y (485 MPa) and f_u (550 MPa) specified by ASTM A1064.

The percentage reduction of area (R_a) exhibited a large scatter of values, mainly due to the difficulty encountered in measuring the diameter of the wires in the failure zone when the samples were deformed or corrugated. ASTM A1064 [23] specifies that the R_a parameter is only considered for smooth wires and can be obtained from the methodology proposed by the ASTM A370 [25] standard. The question arises whether this methodology is suitable for obtaining the R_a parameter for corrugated wires. The results found in this study exhibited large *CV*, indicating that the methodology might not be suitable. At the present time, quality certificates for welded meshes issued by specialized laboratories do not include any determination of the R_a parameter. This may be the result of the absence of any clear methodology allowing this parameter to be obtained in a reliable way for corrugated wires, which are the ones usually subjected to quality tests in structural engineering, as they are the wires used in concrete reinforcement. In general, as the ASTM A1064 and ASTM A370 standards do not provide clear and specific methodologies for reliable estimation of R_a for corrugated wires in electro-welded meshes, it is not possible to consider this parameter as an unquestionable measure of ductility.

The mean values for k (f_u/f_y) in this study were 1.13 and 1.12 for f_y evaluated using the EUL (0.5%) and Offset (0.2%) methods, respectively. The mean values of k were lower than the minimum values required by ACI 318-14 (e.g., k = 1.25) for structures with special energy dissipation capacity. The mean value of the ultimate strain (ε_u) of wires with diameters of 4, 5 and 6 mm varied between 0.82% and 1.46%. The latter values are lower than the minimum deformation capacity established by Eurocode EC-2 (2004) for type A steel (ε_u = 2.5%), which must be used for reinforcement of structures not subjected to seismic demands. Low values for the strain hardening (SH) slope, such as those exhibited by the Colombian wire meshes, may facilitate the concentration of strain demand at few cracks, and thus they are not suitable for seismic-resistant applications. Parameters f_y, f_u and k, along with the ε_u values obtained in this study, suggest that the wires evaluated here were subjected to a disproportionate initial stretching, which produced significant decrease in their strain capacity. This can be corrected if the matrix rod is of a smaller diameter before the cold-drawing process.

The measured tensile strain values of the wires that were measured provide strong technical evidence for the limitations of the use of CD wires as concrete reinforcement of structural elements subjected to high seismic demands with large expected inelastic deformations. With the exception of section 7.3.3.1 of ACI-318-14, where the strain capacity of reinforcement is limited to 0.4% for non-prestressed slabs, the provisions of ACI-318-14 have no explicit requirements for the strain capacity of steel reinforcement. For this reason, it is necessary to establish in NSR-10 and ACI 318-14 a classification for reinforcing steel that is based on expected performance of structural RC members, as does Eurocode EC-2 [21] does. The results here suggest that the WWM currently produced in Bogotá, Colombia should not be used for reinforcement of concrete structures requiring intermediate or special energy dissipation. Use of the wires reported herein as reinforcing steel of concrete members would require careful evaluation of the design hypothesis, as moment capacity may be overestimated. Furthermore, due to the flat post-yield response of the wires, their use in applications in which RC members may undergo large deformations must be restricted. For concrete walls reinforced in the web using the CD steel wire meshes reported herein, the displacement capacity should be close to the story-drift limit of 0.5% prescribed by the Peruvian code for earthquake-resistant buildings, and reductions in the web-steel ratio to increase in yield strength should not be permitted.

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