Evaluation of mechanical and structural behavior of austenitic and duplex stainless steel reinforcements

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Evaluation of mechanical and structural behavior of austenitic and duplex stainless steel reinforcements

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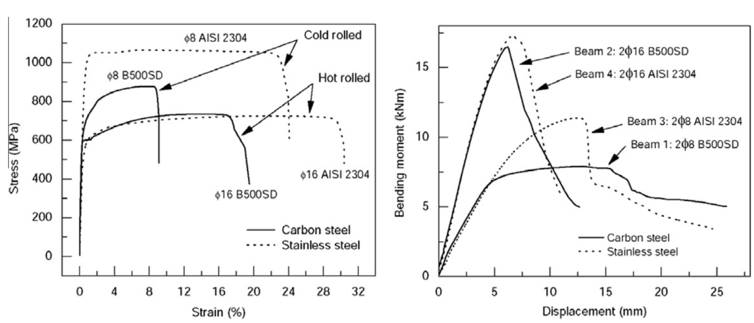
HIGHLIGHTS

- Stainless steel (SS) reinforcements offer high ductility performance to fracture.
- SS reinforced concrete provides long-lasting constructions and buildings.
- SS reinforcements show higher ductility than carbon steel (CS) when hot rolled.
- SS reinforcements reach a slightly lower modulus of elasticity than CS.

GRAPHICAL ABSTRACT



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ABSTRACT

The mechanical and structural behavior of three stainless steel (SS) reinforcing bars (austenitic AISI 304, duplex AISI 2304 and new lean duplex AISI 2001) have been studied and compared with the conventional carbon steel (CS) B500SD. The study was conducted at rebar level, cross-section level and structural member level. The results demonstrate higher ductility performance for hot-rolled SS rebars than for CS, but lower ductility for cold-rolled SS rebars compared to CS. The experimental elastic modulus of SS rebars is lower than that of CS.

1. Introduction

Corrosion of carbon steel (CS) reinforcements is the main durability problem in conventional concrete structures and the most difficult to avoid in certain circumstances. Protection against corrosion is provided by the thickness and the impermeability of the concrete cover, as well as the self-regenerating thin passive oxide layer that is formed at the steel–concrete interphase due to the high alkalinity of concrete. The pH of Portland cement paste during the setting process reaches values ranging between 12.5 and 13.5 because of hydroxide formation.

Steel remains in a passive state until the pH drops to values lower than 11.4 [1], 11 [2] or 9.5 [3]. pH decreases are primarily due either to concrete carbonation (calcium-bearing phases are attacked by carbon dioxide from the air) or the presence of depassivating ions, especially chlorides [4]. The latter may come from salt fog in the case of structures close to the coast, from sea water if they are fully or partially submerged, or from de-icing salts in the case of road bridges in frost areas.

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The feature that distinguishes stainless steel (SS) reinforcements from CS rebars is their excellent resistance to corrosion, including that triggered by chloride ions. SS contains a minimum of 10.5% chromium [5], which forms a very thin self-regenerating chromium oxide layer that is resistant to atmospheric or electrochemical corrosion, as well as other alloys such as nickel, molybdenum, manganese, copper and nitrogen which confer additional features. A wide variety of SS alloys and types can be found for multiple applications.

Austenitic and duplex SS are the two types most commonly employed to manufacture reinforcing bars, although their high price, between 4 and 7 times that of CS [6], limits their use to structures in very aggressive environments or whose life is to be extended. Austenitic SS reinforcements were first used in 1941 at Progreso pier in Mexico [7] and their tolerance to chlorides is 5–10 times higher than that of CS [8]. Duplex SS reinforcements are currently more commonly used because their lower nickel content makes them more economical than austenitic SS, nickel being an expensive alloying element whose market price has suffered strong fluctuations since 2006. Duplex SS rebars also present greater resistance to corrosion by chloride pitting [9,10].

In recent years low-nickel duplex SS reinforcements, which compensate their lower nickel content with a higher manganese content, have appeared on the market. Low-nickel AISI 2001 and 2101 lean duplex SS are cheaper than 2205 and 2304 duplex SS, but all four present similar resistance to chloride corrosion [11,12]. SS rebars offer scope for relaxing concrete durability measures originally designed to protect CS, such as minimum covers [13] and maximum design crack widths [14]. Furthermore, several studies have shown that the combined use of SS and CS rebars does not increase the risk of reinforcement corrosion compared to the use of CS alone, even when the bars are in direct electrical contact [15,16]. This means that the use of SS reinforcements can be restricted to the most exposed structural members in order to lengthen the design service life of newly built structures while reducing their economic impact on the total cost of the concrete structure.

Other interesting applications when cost is not decisive include the rehabilitation of concrete structures affected by corrosion, replacing corroded CS reinforcements or providing a new replacement reinforcement concrete cover [17], and the reinforcement of brick or stone walls on bridges or historic buildings such as churches, cathedrals, etc., placing the highly corrosion-resistant rebars in mortar joints with a minimal cover [18]. In the case of historic buildings – many of which are located in earthquake zones where the structural layout of the walls is adapted to seismic conditions as a result of long experience [19] – the good ductility properties of SS reinforcements are an added advantage.

However, the use of SS reinforcements is still rare and their mechanical and structural behavior is not known in detail, as it is for CS. The present work has studied the performance of SS rebars, a cross-section of a concrete beam and a structural member in order to assess the mechanical and structural behavior of three types of SS reinforcements, one austenitic and two duplex, in comparison with traditional B500SD carbon steel (Spanish EHE-08 standard [20] high ductility and creep-resistant corrugated steel with a yield strength of $f_y \geqslant 500$ MPa, "Grade C" according to Eurocode 2 (EC2) [21]).

2. Experimental procedure

2.1. Study at bar level

The study of the mechanical behavior and ductility of SS reinforcements by analysis of austenitic type AISI 304 (EN 1.4301) and duplex AISI 2304 (EN 1.4362) and 2001 (EN 1.4482) SS together with B500SD carbon steel as a reference has firstly been studied at bar level. These steels have been tested to tensile strength in agreement with European standards EN 10002-1 [22] and ISO 15630-1 [23] using a MIB 60/AM Ibertest machine. The tests were performed on specimens with a nominal diameter of 8 mm in the case of cold-rolled AISI 304, AISI 2304 and B500SD steels, and 16 and 20 mm for hot-rolled steels. Chemical analysis of the studied reinforcements was conducted by inductively coupled plasma emission spectroscopy. Composition results are shown in Table 1.

Based on the results obtained in the tensile tests, ductility parameters have been calculated for each of the four steels according to the following criteria:

(a) The criteria established in several European regulations; specifically Model Code 2010 (MC 2010) [24], and Eurocode 2 (EC2) [21], using the ultimate tensile strength to yield strength ratio f_t/f_y (hardening ratio) and the ultimate strain for the maximum (ultimate) strength ε_u.
(b) The strength requirements and ductility set out in American Standard ASTM A615 [25] for grade 60 steels in the case of calculation for earthquakes.
(c) The concept of equivalent steel according to the p parameter of Cosenza (Eq. (1)), the area A_{nom} defined by Creazza (Eq. (2)) and the toughness index I_d of Ortega (Eq. (3)) [2,26]:

$$p = \left(\frac{f_t}{f_y} - 1\right)^{0.9} (\varepsilon_u + 3\varepsilon_{sh})^{0.75} \quad (1)$$

$$A_{nom} = \frac{2}{3}(\varepsilon_u - \varepsilon_y)(f_t - f_y) \quad (2)$$

$$I_d = \left(1 + \frac{f_t}{f_y}\right)\left(\frac{\varepsilon_u}{\varepsilon_y} - 1\right) \quad (3)$$

where f_t is the ultimate tensile strength, f_y is the yield strength, ε_y is the strain at yield strength, ε_u is the ultimate strain, ε_{sh} is the strain at the end of the yield plateau corresponding to initial strain hardening (for hot-rolled CS reinforcements) and ε_y is the strain at yield strength.
(d) The *Comité Euro-International du Béton* proposal [2] for a new classification of steel ductility based on the formulation of Cosenza, which establishes the following limits for high-ductility or *S*-class steels:

$$\left(\frac{f_t}{f_y} - 1\right)_k \geq 0.13 \quad \text{and} \quad \varepsilon_u \geq 9\% \quad (4)$$

$$\left(\frac{f_t}{f_y} - 1\right)_k \geq 0.15 \quad \text{and} \quad \varepsilon_u \geq 6\% \quad (5)$$

$$\left(\frac{f_t}{f_y} - 1\right)_k \geq 0.17 \quad \text{and} \quad \varepsilon_u \geq 5\% \quad (6)$$

2.2. Study at section level

For the study at section level, moment–curvature M–χ diagrams of two standard beam sections have been produced by iteration of up to seven possible points. The beams were a 50 × 30 cm flat beam and a 30 × 50 cm downstand beam reinforced with different amounts of CS and duplex SS rebars as detailed in Table 2. The duplex SS type was selected because it is the most widely used [27] and exhibits similar mechanical characteristics to the austenitic type, as shown in the tensile test results obtained in the present study.

For calculation purposes, the idealized strength–strain diagrams represented in Fig. 1 have been chosen, taking the parabola-rectangle diagram for concrete compression and the bilinear diagram with a horizontal upper branch for CS and SS rebars. Both diagrams have been prepared according to Eurocode 2 (EC2). A maxi-

Table 1
Chemical composition of tested steels (weight%, balance Fe).

Steel	C	Si	Mn	P	S	Cr	Ni	Cu	N	Mo
AISI 304	0.02	0.28	1.41	0.034	0.023	18.07	7.93	0.33	0.05	0.22
AISI 2304	0.02	0.35	0.81	0.029	0.010	22.75	4.32	0.31	0.14	0.29
AISI 2001	0.03	0.65	4.19	0.023	0.010	20.07	1.78	0.08	0.13	0.22
B500SD	0.20	0.22	0.72	<0.01	0.022	0.13	0.13	0.18	–	–

Table 2
Beam cross-sections considered in the section-level of the study.

Flat beams 50 × 30 cm	Downstand beams 30 × 50 cm	Main reinforcement	Reinforcement quantities per concrete cross-section (‰)
P1	C1	4 Ø12	3.02
P2	C2	3 Ø16	4.02
P3	C3	4 Ø16	5.36
P4	C4	5 Ø16	6.70
P5	C5	6 Ø16	8.04
P6	C6	7 Ø16	9.38
P7	C7	5 Ø20	10.47
P8	C8	6 Ø20	12.57
P9	C9	8 Ø20	16.76

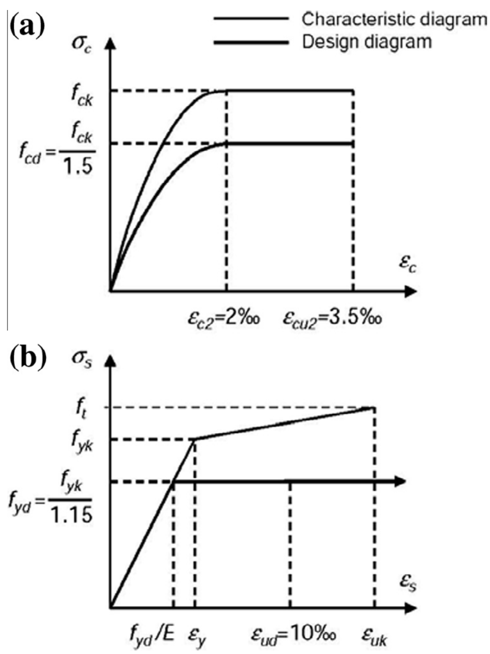

Fig. 1. Characteristic and design strength–strain diagrams of: (a) concrete, (b) carbon steel and stainless steel.

mum steel deformation ε_{ud} of 10‰, the limit established by Spanish EHE-08 standard for structural calculations, has been considered. A specified characteristic compressive strength f_{ck} of 25 MPa has been assumed for the concrete, and a 500 MPa yield strength f_{yk} has been considered for the CS and SS reinforcements – both values being in good agreement with the experimental results. These values have been reduced by 1.50 and 1.15 respectively for the design strength of concrete f_{cd} and steel f_{yd}. The Young's modulus of elasticity E for SS rebars has been obtained in the experimental tensile tests (E = 170,000 MPa, see Section 3.1). The ductility values of cross sections reinforced with both steels have been compared from the M–χ diagrams.

2.3. Study at member level

For the study at structural member level, four concrete beams of 10 × 15 cm section and 1 meter of span length between supports have been prepared (Fig. 2) with the following main longitudinal reinforcements (tension reinforcement):

(a) Beam 1: two CS corrugated rebars B500SD of diameter Ø8.
(b) Beam 2: two CS corrugated rebars B500SD of diameter Ø16.
(c) Beam 3: two duplex SS corrugated rebars AISI 2304 Ø8.
(d) Beam 4: two duplex SS corrugated rebars AISI 2304 Ø16.

The reinforcements were mounted with a Ø8 stirrup placed at 45° and at 7 cm in beams 1 and 3, and at 10 cm in beams 2 and 4. The concrete has a specified characteristic compressive strength of 25 MPa. The beams have been tested to bending up to fracture using the Ibertest machine, applying a load (Q_{max}) at two equidistant points to a third of the beam span length. The behavior to deformation and strength has been studied through the load–deflection relationship.

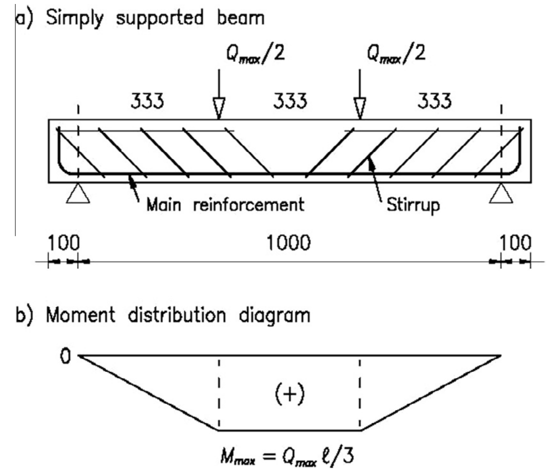

Fig. 2. Dimensions of the beams constructed and tested: (a) simple beam, (b) moment distribution diagram (units in millimeters).

3. Results and discussion

3.1. Reinforcement tensile tests

Fig. 3 shows a comparison of the representative stress–strain curves for each type of the four corrugated steel rebars. The SS presents a curve without a well-defined elastic limit. Standard recommendations for determining the elastic limit with 0.2% proof stress were applied to SS and CS rebars. The tensile test results are shown in Table 3 (mean values of the four specimens tested for each diameter and steel). Testing of the ultimate tensile strength f_t and yield strength f_y reveals differences due to the rebar manufacturing process, either hot or cold rolled, but not on account of the type of steel from which they are produced. The three hot-rolled steel bars present f_y values of between 507 and 609 MPa, so for calculation purposes the nominal value of 500 MPa is suitable for all the tested steels. With regard to cold-rolled rebars (Ø8 mm), the SS bars show 50% higher values than the CS bars.

However, regarding the elastic modulus E, the values obtained are 15% lower in SS reinforcements than in CS rebars, regardless of the rod diameters or rebar manufacturing process. While the value of 200,000 MPa for the elastic modulus of CS is confirmed, a value of 170,000 MPa is obtained for SS reinforcements, which must be taken into account in the structural calculations

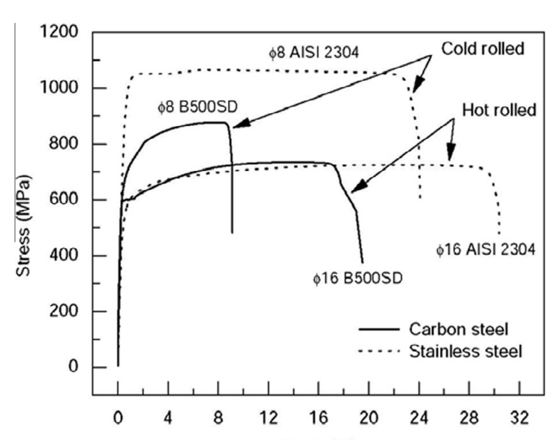

Fig. 3. Comparison of stress–strain curves for stainless steel and carbon steel cold-rolled and hot-rolled rebars.

Table 3
Mean results (and standard deviation SD) of the tensile tests of the three steels: f_t ultimate tensile strength, f_y yield strength, E modulus of elasticity, ε_y strain at yield strength, ε_u ultimate strain and ε_{sh} yield plateau strain.

Steel	Ø (mm)	f_t (MPa)		f_y (MPa)		E (MPa)		ε_y (%)		ε_u (%)		ε_{sh} (%)	
		Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD
AISI 304	8	1060	15	1027	24	145000	3000	1.10	0.10	5.82	0.08	–	–
	16	743	11	548	15	174000	11000	0.55	0.02	18.68	1.44	–	–
	20	728	12	507	9	179000	9000	0.52	0.01	30.12	2.38	–	–
AISI 2304	8	1066	16	1003	29	189000	12000	0.85	0.04	7.02	0.24	–	–
	16	711	8	529	14	179000	4000	0.53	0.02	24.46	2.67	–	–
	20	747	7	609	18	184000	6000	0.57	0.04	25.62	2.75	–	–
AISI 2001	8	733	13	531	16	196000	9000	0.50	0.02	29.70	1.50	–	–
	16	740	15	549	16	177000	7000	0.51	0.01	27.83	2.22	–	–
	20	704	11	536	18	173000	6000	0.58	0.03	28.60	1.25	–	–
B500SD	8	875	13	684	15	216000	5000	0.60	0.04	8.37	0.24	–	–
	16	736	8	602	11	207000	6000	0.54	0.04	13.32	1.44	1.50	0.20
	20	671	4	556	8	206000	3000	0.51	0.03	11.84	1.50	1.09	0.11

Table 4
Experimental parameters of reinforcement ductility (specified characteristic values established by the different regulations considered) and values calculated according to the criteria of equivalent steel for high ductility steel.

Steel	Ø (mm)	f_t/f_y	ε_u (%)	p	A_{nom} (MPa)	I_d
AISI 304	8	1.03	5.82	0.17	113	15
	16	1.36	18.68	3.55	2383	138
	20	1.44	30.12	6.11	4381	257
AISI 2304	8	1.06	7.02	0.36	271	25
	16	1.34	24.46	4.22	2937	193
	20	1.23	25.62	2.98	2321	170
AISI 2001	8	1.38	29.70	5.34	3972	259
	16	1.35	27.83	4.70	3505	208
	20	1.31	28.60	4.41	3177	215
B500SD	8	1.28	8.37	1.56	1025	58
	16	1.22	13.32	2.24	1160	99
	20	1.21	11.84	1.85	885	94
Standard	**High ductility**	$(f_t/f_y)_k$	ε_{uk} (%)	p	A_{nom}* (MPa)	I_d*
MC 2010	D	$\geqslant 1.25 < 1.45$	$\geqslant 8.0$	1.37	388	70
EC-2	C	$\geqslant 1.15 < 1.35$	$\geqslant 7.5$	0.82	363	62
ASTM A615**	Grade 60	$\geqslant 1.25$	$\geqslant 7.0$	1.24	338	61
		$\geqslant 1.13$	$\geqslant 9.0$	0.83	438	75
CEB	S	$\geqslant 1.15$	$\geqslant 6.0$	0.70	288	49
		$\geqslant 1.17$	$\geqslant 5.0$	0.68	238	41

* Calculated for steels with the following minimum values: $\varepsilon_y = f_y/E = 500/200{,}000 = 0.25\%$ and $f_t - f_y = 575 - 500 = 75$ MPa.
** Rebars up to 25 mm in diameter have been considered, with the minimum f_t/f_y ratio that steels grade 40 and 60 must meet in the case of members being subjected to earthquake, according to ACI 318-08 [32].

for reinforced concrete. This result agrees with that obtained by other authors [28–30], even in the case of cold-rolled profiles of austenitic steel [31]. Nevertheless, elastic modulus values may be obtained for different diameters and batches.

3.2. Parameters of reinforcement ductility

Ductility parameters for each reinforcement are summarized in Table 4. For comparative purposes, this table also shows the characteristic values of the $(f_t/f_y)_k$ ratio and of ε_{uk} as established by European and American standards for high ductility CS and the corresponding minimum values of the p Cosenza parameter, A_{nom} Creazza area and I_d Ortega index calculated for each case.

In terms of the hardening ratio f_t/f_y and the ultimate strain ε_u, notable differences have been found between the types of steel and the manufacturing process. In the case of hot-rolled rebars, all the steels meet the minimum ductility requirements established by EC2. However, while CS rebars are very close to these minimum limits – with ratios of 1.21 and 1.22 compared to the minimum of 1.15 – SS rebars greatly surpass these rates, reaching values of up to 1.44. The most notable differences can be seen in the ultimate strain, which in the case of Ø20 rebars is twice as high for SS as it is for CS, with values of between 25.62% and 30.12% compared to 11.84%, respectively.

Nevertheless, in the case of cold-rolled bars the reverse is true, and while carbon steel B500SD meets the ductility requirements of EC2, SS reinforcements do not. Furthermore, hot-rolled CS reinforcements do not meet the strict ductility requirements set in ASTM A615 Standard and stated in MC 2010, because the f_t/f_y ratio remains below the minimum value of 1.25, even if the ultimate strain ε_u values are much higher. In these cases it is interesting to quantify ductility according to the concept of equivalent steel as stated by Cosenza, since this takes into account the combination of both factors.

According to this concept SS rebars reach much higher rates than CS, except again in the case of cold-rolled rebars which reach high resistance but with little elongation. In the particular case of diameter Ø20 AISI 304 SS, the mean p parameter value is 6.11, which means a rotation capacity of 6.11/1.85, i.e. 3.3 times higher than the equivalent CS rebars. However, cold-rolled AISI 304 SS rebars have a mean p parameter of 0.17, in contrast to the 1.56 of CS bars, and so provide a 9.2 times lower rotation capacity.

The ductility classification as proposed by CEB was analyzed, and the results are shown in Fig. 4. The dashed line marks the minimum limit values for the S-class high ductility steel according to Eqs. (4)–(6). The two reinforcements outside the dashed line correspond to cold-rolled SS rebars (diameter Ø8 AISI 304 and 2304 steels). Regarding hot-rolled SS, all the diameters are clearly to the right of their counterparts in CS, showing greater ductility.

3.3. Cross-section level calculation

Relative M–χ diagrams of the flat and downstand beam cross-sections considered in the study are represented in Fig. 5. Relative moment is the correlation of $M/b{\cdot}h^2$, and relative curvature refers to the product $\chi{\cdot}h$. These concepts allow sections of different width-by-depth dimensions $b{\cdot}h$ to be represented in a single diagram. In the diagram, values are marked corresponding to the yielding or elastic curvature χ_e (obtained when steel reaches its yield strength or when concrete has a deformation of 2‰) and the last or ultimate curvature χ_u (when steel reaches an elongation

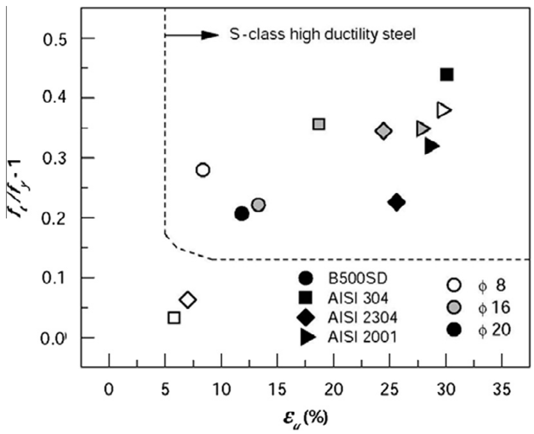

Fig. 4. Ductility classification of *S*-class steel proposed by CEB, and parameters obtained for B500SD carbon steel and AISI 304, 2304 and 2001 stainless steel.

of 10‰ or concrete reaches 3.5‰). Based on the values of χ_e and χ_u the ductility of each cross-section is obtained:

$$D_s = \frac{\chi_u}{\chi_e} \tag{7}$$

and is compared in Fig. 6. It can be seen that D_s values are almost always higher in sections reinforced with CS than when reinforced with SS, with increases of up to 17% in the case of beams P-2 and C-2. This is due to the greater χ_e provided to the SS section, because of its lower elastic modulus E and since the values of χ_u remain equal for both steels, as they are limited in the calculation by the maximum of 10‰ steel strain.

For this reason, despite SS being a more ductile material than CS, it provides for less ductile reinforced concrete cross-sections. The higher ductility of SS compared to CS is shown in some deformations that fall outside conventional structural calculations, but this has a significant advantage in the case of collapse of the structure.

Only for this case, the concept of *curvature to break* χ_{ur} is proposed as the curvature of the RC section when steel reinforcement reaches its actual ultimate strain:

$$\chi_{ur} = \frac{\varepsilon_u}{d - x} \tag{8}$$

where d is the effective depth of the section and x the depth of the neutral axis. From $\chi_{ur,}$ the *fracture ductility of the cross-section* D_{sr} is defined by the ratio (similar to Eq. (7)):

$$D_{sr} = \frac{\chi_{ur}}{\chi_e} \tag{9}$$

D_{sr} values calculated for the studied beams are compared in Fig. 7, assuming they have the same amount of compression reinforcement as stress reinforcement (in which the maximum ductility of the section would be reached). It is proved that the *fracture ductility* of the sections reinforced with SS is between two and

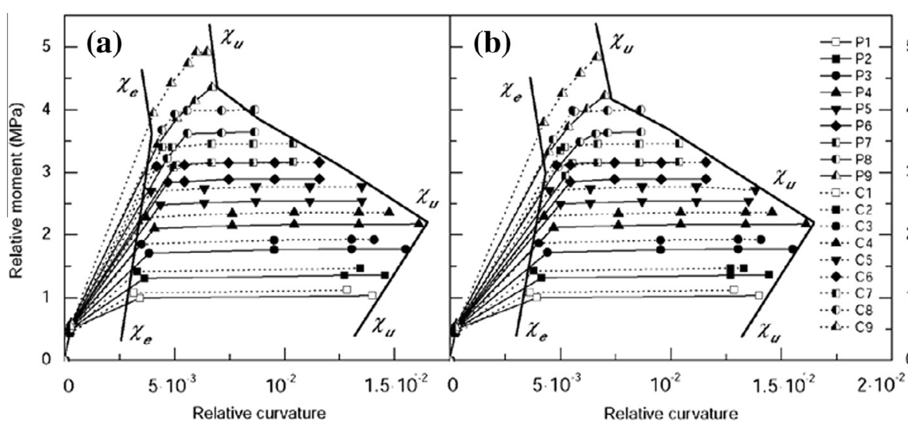

Fig. 5. Relative M–χ cross-section diagrams of reinforced beams with different amounts of: (a) B500SD carbon steel, (b) AISI 2304 duplex stainless steel. Elastic curvatures χ_e and ultimate curvatures χ_u are indicated.

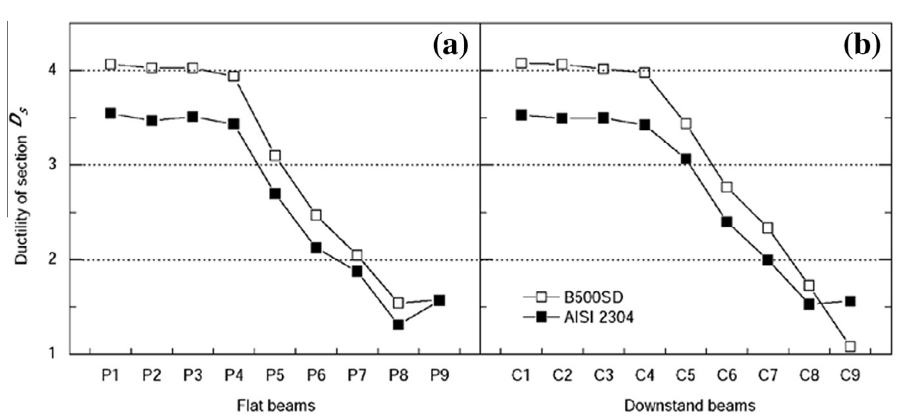

Fig. 6. Ductility of cross-section D_s: (a) flat beams, (b) downstand beams, reinforced with different amounts of carbon and duplex stainless steels.

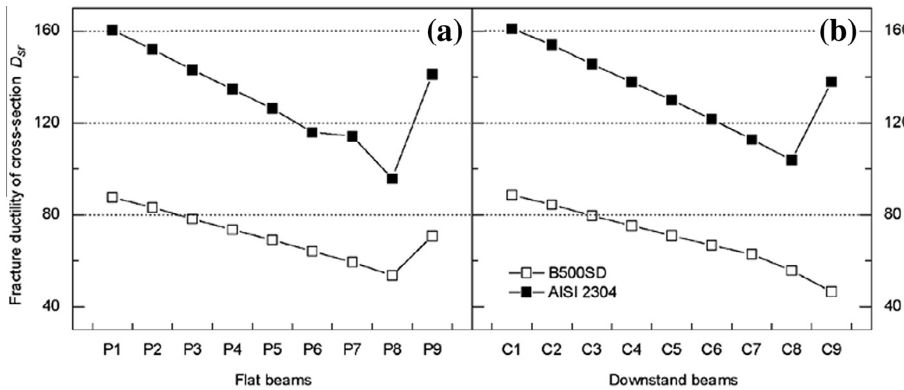

Fig. 7. Fracture ductility of cross-section D_{sr}: (a) flat beams, (b) downstand beams, reinforced with different amounts of carbon and duplex stainless steels, with the same reinforcement both in tension and in compression.

Table 5
Results of the bending tests performed for the four beams.

Beam and reinforced	$f_{c.real}$ (MPa)	Q_{max} (kN)	M_{max} (kNm)	δ_{max} (mm)	h_w (mm)	s_m (mm)
1: 2Ø8 B500SD	26.6	47	7.9	12.9	100	100
2: 2Ø16 B500SD	27.4	99	16.5	6.3	54	110
3: 2Ø8 AISI 2304	26.6	68	11.4	12.6	95	97
4: 2Ø16 AISI 2304	27.4	104	17.3	6.7	70	72

three times higher than those reinforced with traditional CS. Hence this new concept faithfully reflects the excellent ductility properties of SS reinforcements.

3.4. Bending tests of beams

The results of the bending tests of the four beams are summarized in Table 5, where $f_{c.real}$ is the real compressive strength of concrete obtained from specimens prepared to manufacture beams, Q_{max} is the maximum load applied by the testing machine, M_{max} is the ultimate bending moment at the center of the span length ($M_{max} = 0.5 \cdot Q_{max} \cdot \ell/3$), δ_{max} is the maximum deflection marked by the press (displacement at mid-span section), h_w is the height of the highest crack found, measured from the underside of the beam and s_m is the mean separation between cracks appeared during the test.

Fig. 8 shows the moment–displacement diagrams of the four beams. In this figure, the behavior of beam 1 reinforced with 2 corrugated CS Ø8 bars is more ductile than that reinforced with duplex SS, although it reaches a smaller maximum load. However, the behavior is very similar between joists reinforced with two diameter Ø16 corrugated rods, although fracture of the duplex SS reinforced beam is more progressive.

The theoretical calculations of instant deflection δ, fissure opening w and mean separation between cracks s_m, performed for a load Q registered during the test using the formulation contained in instruction EHE-08, lead to the results listed in Table 6. For the calculations consideration has been made of the real strength and elastic modulus values found in the tests for the steels and concrete. Both deflections and theoretical fissure openings correspond to actual measurements in the beam fracture tests, thus confirming the behavior of steels according to their mechanical properties obtained in the tensile tests, especially regarding the lower elastic modulus of stainless steels. However, the mean separation

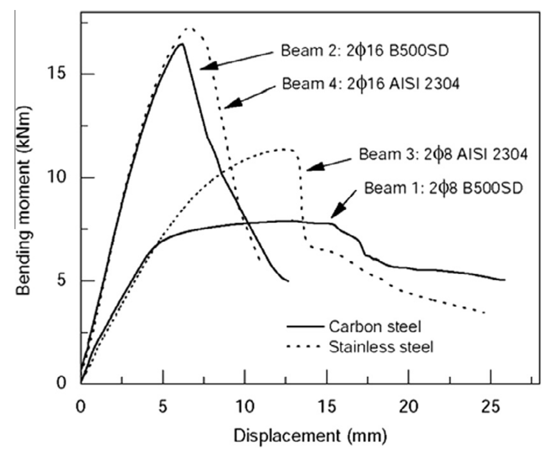

Fig. 8. Moment–displacement diagrams for the four beams tested to ultimate bending.

Table 6
Theoretical calculations of: δ deflection, w fissure opening and s_m mean separation between cracks, in each beam for a given load Q, in compliance with instruction EHE-08.

Beam	Q (kN)	δ (mm)	w (mm)	s_m (mm)
1	40	0.3	0.29	56
2	90	0.1	0.13	44
3	55	0.3	0.48	56
4	95	0.1	0.17	44

between theoretical fissures corresponds to approximately half of the test measurements, a circumstance that can only be related to the characteristics of the concrete and the beam size, since the steel characteristics do not take part in the theoretical calculations.

4. Conclusions

The studied austenitic AISI 304 and duplex AISI 2304 and 2001 stainless steel (SS) reinforcements differ from the B500SD carbon steel (CS) in two important mechanical properties: ductility and modulus of elasticity. The SS reinforcements show higher ductility for hot-rolled rebars, but lower ductility when they are cold-rolled. The SS reinforcements also present a slightly (15%) lower modulus of elasticity compared to the CS rebars.

The high ductility of hot-rolled SS reinforcements is based primarily on the high elongations they reach when subjected to the maximum load in the tensile test, with ε_u values that duplicate those of traditional CS reinforcements. If ductility is quantified according to the concept of equivalent steel, or at the fracture level of the concrete section (Eqs. (8) and (9)), then the values of SS reinforcements triple those of CS. This high ductility has advantages not only in cases of collapse of the concrete structure, but also in the increased energy the hot-rolled stainless steel reinforcements are able to absorb, making them especially suitable for structures located in seismic zones – both new structures and in renovations and restorations. SS reinforcements offer high ductility performance to fracture, therefore enabling plastic hinge regions with higher cross-section rotation capacity than traditional CS rebars.

The results obtained show the importance of studying the elastic modulus value for different diameters and batches. In the present study a lower elastic modulus is exhibited for SS reinforcements. Thus, when designing both deflection calculations and crack width prediction it is necessary to consider larger amounts of steel. In practice, this increase is offset by the savings that are achieved in terms of reducing the amount of reinforcement necessary when a thinner cover needs to be used. This leads to an increase in the maximum design crack width withstood by the high corrosion resistance of SS rebars.

Despite the higher cost of SS reinforcement compared to CS rebars, SS is highly recommended for use in sea-front areas with extremely aggressive environments. The reason for this is that the reinforced concrete service life is increased, thus providing long-lasting constructions and buildings.

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