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Study of the Residual Bond Strength between Corroded Steel Bars and Concrete—A Comparison with the Recommendations of Fib Model Code 2010

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Abstract: As is well known, corrosion of steel reinforcement deteriorates the steel–concrete interface and causes concrete cracking, degrading significantly the bond strength. Several experimental studies have investigated the magnitude of residual bond strength due to corrosion, which affects either the function of corrosion-damaged steel bars or the surface crack width in concrete. As a result, linear and exponential correlation relationships have been proposed in order to predict the bond loss due to corrosion. Based on the results of an ongoing experimental campaign on the degradation of bond strength of RC specimens, combined with comparable outcomes from existing literature, this manuscript summarizes a database, comparing with the recommendations of Model Code 2010, to analyze and interpret the corrosion effect on the bond loss and highlights some points that need improvement in the current regulations. As indicated, the density of transverse reinforcement (stirrups spacing) has intense impact on the resulting bond loss due to corrosion. Hence, in order to quantify this aspect, the present manuscript introduces a discretization of confinement levels of RC elements, depending on the stirrups spacing. Based on this, regression analyses of data were conducted to extract fitting curves of bond loss, taking into account the amount of transverse reinforcement and predictive zones of residual bond strength in relationship to either corrosion penetration or surface crack width. Furthermore, the outcomes demonstrate that the corrosion penetration depth is an appropriate assessment tool to correlate the residual bond strength with the corrosion level, whereas the surface crack width on concrete is not yet an effective index, since there is a plethora of factors affecting the crack width. Due to this, more research is needed to improve the current level of knowledge on the surface crack width and link it with the corrosion damage of the steel bar and the residual bond strength due to corrosion.

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1. Introduction

Reinforced concrete (RC) is the most widely used construction material of existing building stock, providing high bearing capacity and ductility, compared to its low production costs. However, the durability of RC structures is adversely affected by the corrosion of steel reinforcement, since it degrades the mechanical performance of both the steel bars and concrete and, mainly, the bond mechanism between them, which plays a key role in ensuring their proper interaction. From a financial perspective, the magnitude of corrosion problem has been highlighted by various studies and has been marked by its enormous costs for repairing and rehabilitating the damaged structures [1–3]. For instance, a report conducted by the National Association of Corrosion Engineers (NACE) [2] indicates that the cost of corrosion is estimated globally to be USD 2.5 trillion, or approximately 3.4 percent of the global gross domestic product (GDP).

Corrosion of steel reinforcement is an electrochemical phenomenon, in which steel tends to return to its original form (ore) by forming iron oxides on its surface (rust). As shown in Figure 1, the oxides produced on the steel surface occupy a greater volume than the parental steel mass [4], which is lost, causing tensile stresses in the surrounding concrete, therefore leading to cracking and, finally, spalling of the cover thickness. As is well known, the corrosion damage of embedded (in concrete) steel reinforcement due to chloride penetration is mostly defined as nonuniform, resulting in diverse reduction of the cross sectional area along the length of the steel bars. Due to this uneven distribution of corrosion phenomenon on steel surface, increase of applied stresses occur within the material, affecting negatively the entire bearing capacity and ductility of RC elements. In this framework, several researchers have investigated the corrosion phenomenon and its abovementioned detrimental consequences on the mechanical response of steel reinforcing bars [5–10]. Nevertheless, prior to the degradation of mechanical properties of steel reinforcement, corrosion initially influences the steel–concrete interface, allowing the development of relative slip and reducing the bond strength between them. From this point of view, the prolonged exposure of RC structures to a corrosive environment affects the bond mechanism between steel and concrete and degrades the bearing capacity in anchorage zones, and the rotational capacity in plastic hinge regions. As a result, the bond loss effect due to the environmental action prevents the development of full bearing capacity of reinforced concrete elements. The importance and influence of corrosion phenomenon on the load-bearing capacity of RC elements has been indicated by many researchers [11–15]. The experimental results have demonstrated the detrimental consequences of corrosion of steel reinforcement on the service life of RC elements, leading to reduction of their load-bearing capacity and, mainly, to limit their rotational capacity, which leads to brittle failure, when compared to noncorroded conditions [11–15].

Figure 1. Cracking development and spalling of concrete cover due to corrosion of steel reinforcement, reproduced from [16].

Taking into account that bond mechanism is one of the main criteria in design of RC members, it is obvious that the estimation of the residual bond strength due to corrosion is considered necessary in order to determine the durability of RC elements, both in new modern designed structures and in existing aging ones. Many experimental studies have been conducted in order to estimate the effect of reinforcement corrosion on bond loss and to investigate its influencing factors, such as the concrete cover thickness, the density of stirrups, the surface concrete cracking, the geometry of ribs on rebar's surface and the concrete class [16–24].

Furthermore, several researchers have carried out experiments on corroded RC elements, via pull-out tests or beam-end tests, so as to link their residual bond strength with the corrosion level of reinforcement, in terms of either the percentage mass loss of bars or the corrosion penetration on their circumference [21,25–27]. These terms of corrosion damage are easily measured in laboratory conditions either by gravimetric measurements

or optical scanning techniques. However in actual structures, the estimation of corrosion level of steel bars presents a difficult task, since they are embedded in concrete and are inaccessible. In addition, due to the fact that corrosion damage is nonuniform on the steel surface, individual measurements on discrete regions may lead to misleading estimations. These difficulties have motivated the scientific community to investigate a possible correlation between corrosion damage of steel bars and surface concrete cracking. More specifically, cracks due to steel corrosion on concrete surfaces are visible and their width can be easily measured, thus providing a useful nondestructive assessment tool [28]. Fischer and Özbolt demonstrated that crack width is an appropriate indicator for bond degradation and represents it better in comparison with the average corrosion penetration [29]. In this framework, recent scientific studies tend to quantify the corrosion damage of the embedded steel reinforcement through the surface cracking width and, subsequently, estimate the local bond loss [16,17,25–32].

To date, several studies have proposed analytical models in order to evaluate the radial pressure due to corrosion oxides and subsequently to predict the loss of initial bond strength between steel and concrete due to the corrosion effect [26,33–35]. Coccia et al. developed an analytical model, which provides the residual bond strength due to corrosion related to the mass loss of steel bar, representing the unconfined conditions (without stirrups) [27]. Chen et al. presented an analytical model for bond degradation of a corroded reinforcing bar, including the influence of adhesion, confinement and corrosion level [35]. Following this approach, predictive models of residual bond strength in relation to the average crack width have been proposed [16,21,36]. It is widely noted that there is a slight increase of bond strength at levels of corrosion up to 2%, followed by an exponential decrease in bond strength. This fact is typically associated with the development of surface splitting cracks [25]. However, outcomes of the various experimental studies are characterized by scattering and diversity due to several influencing factors of both accelerated corrosion and bonding tests.

Nonetheless, while the theoretical framework is well-established, the reliable prediction of the corrosion process in the RC structural members has hardly been successful due to the inherent uncertainties. The current knowledge is based on experimental bond resistance data of corroded RC specimens, where the accelerated corrosion technique and the methods used to test bonding differ. This is depicted in the existing recommendations of Model Code 2010 [37], where indicative ranges of residual bond strength due to corrosion are proposed, as a function of corrosion penetration depth and surface cracks, taking solely into account the presence or absence of stirrups (links) as an on–off criterion. However, the magnitude of bond loss due to corrosion is highly affected by the passive confinement level through the amount of transverse reinforcement. Hence, there is need to amend the international design codes regarding the quantification of residual bond strength due to corrosion. In addition, the penetration depth as a corrosion damage index refers to a uniform reduction of the rebar cross section, whereas the damage is mainly depicted by nonuniform pitting corrosion. Due to these factors, research is needed to establish degradation factors, including the effect of both the density of stirrups and local reduction of cross-section in maintenance of bond strength of corroded RC elements. Based on broad and ongoing experimental research related to the bond loss of corroded RC specimens, combined with collected experimental data in literature, this manuscript investigates the degradation laws of bond strength due to corrosion and focuses on the factor of confinement through stirrups density. Furthermore, the recollected experimental results are compared to the recommendations of Model Code 2010 [37], to demonstrate the gap of the existing design codes and to enhance them.

2. Design Bond Strength Proposed by Model Code 2010

2.1. Design Bond Strength f_{bd}

Bond is the term used to denote the interaction and transfer of force between reinforcement and concrete, as referred in Model Code 2010 [37]. Bond influences structural performance of concrete elements and depends on several factors, namely rib geometry, concrete class and concrete cover, among others [37].

The basic bond strength $f_{bd,0}$, proposed by Model Code 2010 [37], is the average stress on the surface of rebar and is calculated by Equation (1):

$$f_{bd,0} = \eta_1 \cdot \eta_2 \cdot \eta_3 \cdot \eta_4 \cdot (f_{ck}/25)^{0.5} / \gamma_c \quad (1)$$

The value of basic bond strength $f_{bd,0}$ depends solely on steel rebar (morphology of surface, casting conditions, bar diameter and technical class) and on concrete class. More specifically, η_1 stands for the type of ribbed bars (including galvanized and stainless reinforcement) or epoxy coated ribbed bars, η_2 stands for the position of the bar during concreting, η_3 stands for the bar diameter and η_4 stands for the yield stress of steel reinforcement class. Moreover, the influence of concrete on bond strength is taken into account through the value of compressive strength f_{ck} , combined with its safety coefficient γ_c .

Furthermore, in order to take into account the rebar's confinement due to concrete cover and transverse reinforcement, design ultimate bond strength f_{bd} is proposed and determined by Equation (2):

$$f_{bd} = (\alpha_2 + \alpha_3) \cdot f_{bd,0} - 2 \cdot p_{tr} / \gamma_c \quad (2)$$

where α_2 accounts for the influence of confinement due to concrete cover thickness, α_3 accounts for the influence of confinement due to transverse reinforcement (stirrups) on bond strength, respectively, and p_{tr} is the mean compression stress perpendicular to the potential splitting failure surface at the ultimate limit state (and is negative when transverse stress is negative).

For the case of ribbed bars, the coefficient α_2 is determined by Equation (3):

$$\alpha_3 = k_d \cdot (K_{tr} - a_t/50) \geq 0.0, K_{tr} \leq 0.05 \quad (3)$$

2.2. Degradation of Bond Strength Due to Corrosion Effect

Corrosion phenomenon is responsible for degradation of mechanical properties of steel reinforcement along with the reduction of its cross-sectional area, leading to cracking and spalling the cover concrete. Herein, the adverse effects on the steel–concrete interface bring about bond loss between the two components. Model Code 2010 [37] proposes, as depicted in Table 1, the degree of reduction in residual bond strength for corroded reinforcement, in relationship to either corrosion penetration or with the surface crack.

Table 1. Magnitude of reduction in residual bond strength for corroded reinforcement (Fib Model Code 2010).

Corrosion Penetration (mm)	Equivalent Surface Crack (mm)	Confinement	Residual Capacity (as % of f_{bd})	
			Ribbed	Plain
0.05	0.2–0.4	No links	50–70	70–90
0.10	0.4–0.8		40–50	50–60
0.25	1.0–2.0		25–40	30–40
0.05	0.2–0.4	Links	95–100	95–100
0.10	0.4–0.8		70–80	95–100
0.25	1.0–2.0		60–75	90–100

3. Experimental Study of Bond Loss Due to Corrosion

A total of 48 prismatic concrete specimens of dimensions $240 \times 200 \times 310 \text{ mm}^3$ with concrete class C20/25 were casted, in which a main reinforcing steel bar, of technical class B500c and nominal diameter equal to 16 mm, was eccentrically placed on the top of each specimen. The specimens were divided into two groups, with concrete cover equal to 25 mm or 40 mm, as the case may be. Moreover, each group was subdivided into four groups, just as many as the cases of the tested number of transverse reinforcement of nominal diameter equal to 8 mm. The four subcategories examined were (a) specimens without stirrups (N), specimens with wide stirrups $\Phi 8/240 \text{ mm}$ (S240), specimens with stirrups $\Phi 8/120 \text{ mm}$ (S120) and specimens with dense stirrups $\Phi 8/60 \text{ mm}$ (S60). For each combination of concrete cover and stirrups spacing, noncorroded specimens and five corroded specimens were tested. The target corrosion levels of the five specimens were 1%, 2%, 4%, 6%, and 8%. All parameters of the experimental program that were tested are summarized in Table 2.

Table 2. Test matrix of the specimens.

Concrete Cover (mm)	Stirrups Spacing (mm)	Target Corrosion Level (%)
25 (A)	0 (N)	0 (Noncorroded), 1, 2, 4, 6, 8
	240 (S240)	
	120 (S120)	
	60 (S60)	
40 (B)	0 (N)	0 (Noncorroded), 1, 2, 4, 6, 8
	240 (S240)	
	120 (S120)	
	60 (S60)	

Furthermore, the main reinforcing steel bar of each specimen was exposed to corrosion at its central section with a length equal to 250 mm and the rest was protected with a wax layer, whereas all stirrups were protected with a special anticorrosive coating (epoxy resin), in an attempt to isolate the corrosion to only the main reinforcing steel bar. Figure 2a shows the typical geometry of specimens (cross-sectional and longitudinal view), and Figure 2b shows the installation of steel reinforcement in the molds prior to concreting.

Figure 2. (a) Typical geometry of specimen and (b) installation of steel reinforcement in the molds.

Before mixing the concrete in molds, all reinforcement (steel bar and stirrups) were aligned and fastened to the molds. For the concrete mixture, a Portland cement was used, with a water–cement ratio of 0.55 and 20 mm maximum-size coarse aggregate, where the compaction of concrete was conducted via a table vibrator. After the casting, all specimens remained for three days in a wet environment; the molds were then carefully removed and the curing of specimens took place for 28 days in a specially designed room under a temperature of 22°C and continuous wetting. In parallel, plain concrete cube specimens of 200 mm^3 were casted; after 28 days, the average value of compressive strength was evaluated at $f_c = 30 \text{ MPa}$.

3.1. Accelerated Corrosion Technique

The occurrence of corrosion phenomena of actual reinforced concrete structures in the natural environment takes place slowly over a period of decades. In order to accelerate the consequences of natural corrosion, laboratory experimental tests on the reinforced concrete specimens were carried out by the anodic corrosion method (Figure 3a,b). With a view to simulate the corrosion process and to probe the effect of different intensities of corrosion phenomena on bond loss, the specimens were retained in the cells, filled with 5% sodium chloride (NaCl) solution by the weight of water (for different exposure times) and a constant current of 0.12 A was induced via a continuous power supply. The imposed current to the main reinforcing bar of 250 mm corresponds to a current density of about 1 mA/cm². However, in the case of a group of specimens with stirrups, the increased volumetric ratio of steel resulted in decreased imposed current density to the longitudinal bar. Several studies have indicated the difference between the theoretical mass loss via Faraday law and the actual mass loss from the experiments [38]. Following that, the time necessary for obtaining the target corrosion levels (1%, 2%, 4%, 6% and 8% mass loss of the longitudinal bar) could not be accurately estimated through the theoretical Faraday law, since the steel bars were embedded in concrete and the different amount of transverse reinforcement (stirrups spacing) alters the corrosion rate. Thus, based on previous scientific works [30], which correlate the corrosion level of steel bars with the width of surface cracking on concrete, the end of each accelerated corrosion time was set until a desired crack width developed.

Figure 3. (a) Schematic illustration of the accelerated corrosion technique. (b) Accelerated corrosion test of the RC specimen.

Taking into account the study by Lin et al. [39], which associated the bond reduction relative to corrosion of the transverse reinforcement, all stirrups were protected with a special anticorrosive coating (epoxy resin), in order to sufficiently maintain their corrosion resistance. Hence, the degradation is limited only in the longitudinal reinforcing bar and the results of the pull-out tests are not additionally affected by corrosion of the stirrups. Nonetheless, a significant part of the applied current is consumed simultaneously by the protective epoxy coat of the transverse reinforcement. Thus, the current density for each one of the four cases of stirrup spacing does not give the same value, leading to different exposure times required to obtain the target corrosion level. This fact is fully in compliance with actual structures, where different values of current density are reported, due to the different amount of steel reinforcement.

3.2. Mapping of Cracks

Cracks appeared on concrete surfaces due to corrosion of steel reinforcement, mainly on the upper side of specimens and in a direction clearly parallel to the main longitudinal reinforcing bar, as shown in Figure 4a. The average width of these cracks was mapped, measured and calculated (Figure 4b) in order to estimate the corrosion damage through concrete cracking and, subsequently, to correlate this cracking with bond strength.

Figure 4. (a) Cracking development on concrete surface during accelerated corrosion test. (b) Mapping and measurement of surface crack width due to corrosion.

3.3. Pull-Out Tests

Upon the completion of corrosion experiments and mapping of surface cracks, pull-out tests were carried out, based on ASTM C234-91a [40]. As shown in Figure 2, the main (semi-embedded) reinforcing bar was casted eccentrically, on the top of the RC section; however, the specimens were set in a specially designed apparatus (Figure 5a), which enables only uniaxial loading during the pull-out tests, preventing the development of bending moments. All specimens were loaded under displacement-control at a constant displacement rate of 0.4 mm/min, so as to quantify the bonding resistance, which is expressed in Equation (4):

$$f_{bd} = \frac{F_{max}}{\pi \cdot D \cdot L_{emb}} \quad (4)$$

where F_{max} is the maximum pull-out load; $D = 16$ mm, the nominal diameter of the main reinforcing bar; and $L_{emb} = 250$ mm, the embedded length. Moreover, the maximum pull-out load F_{max} and the corresponding normalized bond strength $f_{bd,cor}/f_{bd,non-cor}$ were recorded.

Figure 5. (a) Schematic procedure of the pull-out test. (b) Pull-out test of the steel reinforcing bar.

4. Results and Discussion

After the completion of the experimental procedure, values of corrosion level (in terms of percentage mass loss of the main reinforcing bar), average crack width of surface concrete cracking and normalized bond strength were extracted. The experimental results and correlation relationships between them can be found in detail in previous works of the authors [16,17].

4.1. Normalized Bond Strength versus Corrosion Damage

Based on these data, bond loss between the corroded steel reinforcing bar and concrete as a function of corrosion level is depicted in Figure 6, coupled with comparable data from literature. Furthermore, in order to compare the collected database with the recommendations of Model Code 2010 [37], bond loss values are presented in Figure 7, as a function of corrosion penetration.

For this purpose, the collected values of percentage mass loss of steel bars were expressed in terms of corrosion penetration (depth), using Equation (5) [25]:

$$x = \frac{d_b}{2} (1 - \sqrt{1 - \eta}) \quad (5)$$

where x is the corrosion penetration depth (in mm), η is the corrosion level in terms of percentage mass loss and d_b is the nominal diameter of the steel bar (in mm).

Following the recorded values in both Figures 6 and 7, it can be understood that the gradual increase of corrosion damage leads to bond loss, degrading the initial bond strength (in noncorroded conditions). However, it is obvious that there is no clear relationship between bond loss of RC specimens related to the corrosion damage, either in terms of mass loss or corrosion penetration, as a large scatter of data is demonstrated. This scatter is due to the fact that the rate of bond loss not only depends on the corrosion damage of the reinforcing bars, but more factors are involved.

Figure 6. Normalized bond strength due to corrosion as a function of corrosion level (percentage mass loss) of the tensile reinforcing bar, data from [16,17,21,23,26,41–45].

Figure 7. Normalized bond strength due to corrosion as a function of corrosion penetration depth of the tensile reinforcing bar, data from [16,17,21,23,26,36,37,39,41–46].

The different experimental corrosion techniques and their parameters have an impact on the extracted results. More specifically, the magnitude of imposed current density on steel bars affect significantly the resulting quality of corrosion damage on them, as previously indicated by several researchers, e.g., Andrade, Apostolopoulos, Tondolo et al. [30,47,48]. Moreover, the abovementioned current density combined with the exposure conditions of specimens (continuous wetting, wet/dry cycles, fully submerged specimens) influence the formation and subsequent propagation of cracks in the concrete cover, which is the main factor influencing bond strength. As recently indicated by Mak et al. [25], the liquid corrosion products, which are formed due to limited oxygen in wet exposure conditions, flow outward through the voids and porous structure of the concrete, before solidification. Therefore, corrosion-induced cracking can be delayed or not occur, since expansive pressures are relieved [25]. Due to the influence of corrosion exposure conditions, there is an inconsistency in the correlation between the corrosion damage of steel bars and the surface crack width on concrete. Given that, Model Code 2010 proposes correlation ranges between the depth of corrosion penetration and surface crack width on concrete (Table 2); the proposed values should be interpreted with caution, taking into account the exposure conditions.

The short embedded length of reinforcing bars, as recommended by Rilem [49], is also a crucial parameter of the bond testing. The majority of experimental bond tests are conducted on small sized cubic or cylindrical concrete specimens, where the tested steel reinforcing bar is set at the center. The choice of short embedded length in bond tests is followed so as not to yield the tested steel bar and to determine the local bond stress–slip relationship. However, the contribution of confinement through stirrups cannot be investigated and taken into account due to the limited size of specimens. Hence, this type of specimen and bond tests cannot precisely represent the conditions in actual reinforced concrete elements, where the main reinforcing bars are confined in a great level through transverse reinforcement (stirrups) and are placed eccentrically in the cross-sections. Furthermore, in the case of short embedded length and subsequently short corroded length, the various influencing factors of bond strength, such as concrete strength and cover thickness, and the existence of concrete cracking may predominate and overlap the adverse effect of corrosion, leading to the abovementioned dispersion of experimental results (Figures 6 and 7).

As noted in the previous section, there is a general consensus that concrete cover thickness and stirrups contribute significantly to the bond strength through confinement. Observing the points of Figures 6 and 7 (shown in black), representing the unconfined specimens (without stirrups), rapid bond loss is recorded as the corrosion damage increases. This results from the corrosion-induced cracking of cover thickness, which is the main factor of confinement in the case of no stirrups. Thus, cracking of concrete cover due to steel corrosion weakens the passive confinement level in this group of specimens, leading to bond loss from the very early levels. Therefore, it can be considered that the parameter of cover thickness could be deemed to be negligible in the case of cracked and corroded RC specimens.

Contrary to unconfined concrete, the presence of stirrups and their density play an important role in the rate of degradation of bond strength between corroded steel bar and concrete, providing sufficient confinement in the core of sections. Taking this fact into consideration, it is worth noting a comparison of the experimental outcomes, extracted by confined specimens, with the recommendations of Model Code 2010. As presented in Table 1 of the previous section, Model Code 2010 distinguishes two cases of bond strength degradation depending either on the presence of stirrups (links), neglecting their density, or on their absence (no links), and proposes lower and upper boundaries for their residual bond strength due to corrosion. It is obvious from Figure 7 that the prediction behavior according to Model Code 2010 for unconfined specimens (no links) is in a good agreement with the experimental results and on the safety side, proposing rapid loss of bond strength. On the other hand, in the recommendations of Model Code 2010 there is an ambiguity in case of presence of stirrups (links), which simulate the actual RC elements, since the stirrups density is not taken into account. Due to this fact, an overestimation or underestimation is depicted at values of bond loss, owing to the different levels of confinement in the tested specimens.

For this purpose, in the present study, an attempt is made to juxtapose all experimental results in relation to the presence and subsequently to stirrups density. Herein, four subgroups are proposed depending on the confinement level, as shown in Table 3.

Table 3. Proposed groups of confinement level depending on the stirrups spacing in RC elements.

A/A	Confinement Level	Stirrups Spacing S (mm)
Group A	Unconfined (No stirrups)	Absence or $S > 300$
Group B	Slightly confined	$300 \leq S < 200$
Group C	Moderate Confined	$200 \leq S < 100$
Group D	Narrow confined	$S \leq 100$

Taking into account the different ranges of confinement level in accordance to the abovementioned aggregation of Table 3, regression analysis of experimental data was performed in order to extract predictive curves for each group of confinement. All recorded values of normalized bond strength greater than one were ignored, in order to take into account solely the negative impact of corrosion on bond strength and, subsequently, extract degradation laws. Following previous studies of several researchers, who have indicated the exponential degradation of bond strength toward one that is linear, exponential fitting curves are selected, the function of which is described in Equation (6):

$$\frac{f_{bd,cor}}{f_{bd,non-cor}} = e^{-A \cdot x} \quad (6)$$

where A is a parameter that depends on the stirrups density in the RC element and x is the corrosion penetration depth (in mm).

The outcome of regression analysis for each confinement group is depicted in Figure 8, along with the value of parameter A and the corresponding value of the R^2 coefficient, which are summarized in Table 4. The extracted diagrams in Figure 8 include experimental data, which were functionally separated with reference to the confinement level, in accordance with the proposed subgroups, as shown in Table 3.

Figure 8. Extracted exponential fitting curves of bond loss as a function of corrosion penetration depth, for each confinement level group. (a) Unconfined specimens, (b) Slightly confined specimens, (c) Moderate confined specimens, (d) Narrow confined specimens.

Table 4. Parameters by regression analysis for the exponential predictive model.

	Group A Unconfined	Group B Slightly Confined	Group C Moderate Confined	Group D Narrow Confined
A	5.16	3.82	1.47	0.90
R ² (%)	79	80	69	92

The specific exponential function was chosen due to its simplicity, since there is only one tested parameter, which refers to the density of stirrups in RC elements. As demonstrated by the results of regression analysis in Table 4, the value of parameter A is reduced as the confinement level (number of stirrups) increases; thus, it can be quantified directly with the volumetric ratio of transverse reinforcement. In this manner, parameter A could be a useful assessment tool for the ductile behavior of RC elements, especially in anchorage zones, where the confinement level is critical.

As presented before, Model Code 2010 refers to the residual bond strength due to corrosion in the presence or absence of stirrups. In the case of absence of transverse reinforcement (no links), in Figure 8a, the Model Code 2010 recommendations satisfactorily predict the normalized bond strength of the corresponding tested specimens of group A (no stirrups). On the other hand, in the case of presence of transverse reinforcement (with links), there is a gap in the predictive curves, owing to the different number of stirrups in RC specimens. More specifically, those proposed by Model Code 2010 boundaries tend

to predict accurately the residual bond strength only of group C (moderate confinement), shown in Figure 8c, whereas it gives a conservative estimate for group D (dense stirrups) (Figure 8d). Groups C and D represent modern RC structures designed by demanding codes. Moreover, as depicted in Figure 8b, the predictive boundaries of Model Code 2010 overestimate the bond loss of slightly confined RC elements, group B, which are indicative of existing aging structures with wide stirrups spacing. Finally, according to the experimental data, the predictive boundaries for all cases need to be extended for greater levels of corrosion damage.

In this framework, the present study aims to overcome this knowledge gap regarding the prediction of normalized bond strength due to corrosion, focusing on the impact of the amount of transverse reinforcement (stirrups density). For this purpose, the extracted lower and upper boundaries of the previously mentioned predictive exponential curves for each case of confinement level (Figure 8) are utilized, in order to propose discrete zones of bond loss prediction, depending on both corrosion of steel bars and stirrups density in RC elements. Hence, three different cases are proposed, as shown in Figure 9. The upper red zone (Group D) refers to the case of dense stirrups, where a slow rate of bond loss between corroded steel reinforcement and concrete is depicted. This occasion represents critical regions of RC members, which are designed by modern regulations, resulting in the increase of their bearing and rotational capacity. Therefore, the maintenance of the initial bond strength in those regions is crucial for the overall structural behavior. In addition, the green zone (Group C) is introduced to classify RC members with a moderate confinement level, such as the midspan of beams, in which the corrosion damage affects the bond mechanism between steel and concrete; however, the presence of stirrups influences the ratio of bond loss. The third proposed zone shown in gray (Groups A and B) refers to insufficient confinement level, due to the absence of stirrups or wide spacing between them. In those cases, corrosion damages the interface between steel and concrete and leads to cracking of the concrete cover, leading to rapid bond loss. It is noteworthy that the latter zone simulates the majority of existing RC buildings, in which the use of wide stirrups, namely four pieces per linear meter ($\Phi 8/250$ mm), was followed.

Figure 9. Proposed boundaries of bond loss depending on confinement level.

4.2. Corrosion Penetration Depth as an Index of Corrosion Damage

Corrosion penetration depth is the most widely used index of corrosion damage on steel reinforcing bars, as used in scientific research and current regulations and standards

(i.e., Model Code 2010). The vast majority of scientific studies use the corrosion penetration depth, assuming a homogenous distribution of corrosion damage along the length of the reinforcing bar. However, the corrosion process in actual reinforced concrete structures takes place in limited regions of the embedded reinforcing bars (anode regions), where high amounts of oxygen are concentrated, causing localized reduction of the bar diameter. Several experimental studies on the corrosion damage of reinforcing bars have indicated different types of damage, in the case of embedded or bare steel bars. More specifically, in the framework of this experimental study, the embedded corroded steel bars display large pits and localized reduction of cross-section, in locations of cracking development and, especially, on contact points of longitudinal and transverse reinforcement, as illustrated in Figure 10. The nonuniform corrosion damage and the crucial characterization of failure pits can be accurately measured in laboratory through gravimetric or scanning techniques. However, in actual structures, steel bars cannot be extracted, since they are embedded in concrete, and estimating their corrosion level is a difficult task. Moreover, the corrosion damage is depicted in a nonuniform way on the steel surface. This fact makes the task of evaluating the damage nonrepresentative, since the individual measurements may involve significant probability of error and can lead to nonrepresentative estimations. Hence, an issue arises upon the reliability level of the assessment of corrosion damage via the corrosion penetration depth index.

Figure 10. Pitting corrosion of embedded steel reinforcing bars and localized reduction of cross-section.

Furthermore, corrosion penetration depth should be approached with caution and due consideration, as different levels of percentage mass loss are observed in steel bars of different nominal diameter. In that manner, as derived from Equation (5), different levels of percentage mass loss are exhibited for equal level of corrosion penetration depth, namely, a steel bar of nominal diameter equal to $\varnothing 8$ displays percentage mass loss of 7.4%, whereas a steel bar of nominal diameter equal to $\varnothing 16$ displays percentage mass loss of 3.7%, respectively.

4.3. Normalized Bond Strength Versus Crack Width

The assessment of corrosion damage on steel reinforcing bars presents a difficult task in engineering. Most experimental studies use the abovementioned terms of percentage mass loss or the corrosion penetration depth in order to estimate their corrosion level. These adopted terms of damage are easy to calculate in the laboratory, by gravimetric measurements or techniques for scanning the steel specimens. However, in actual reinforced concrete elements, these methods cannot be followed, since the reinforcing steel bars are embedded in concrete, in combination with the nonuniform damage due to corrosion. Thus, nondestructive methods are used in order to estimate efficiently the damage of steel bars.

As previously indicated by the authors, the surface crack width on concrete seems to be an appropriate nondestructive assessment tool of corrosion, linking the measured crack width with the corrosion damage of reinforced concrete. This agrees with the results of other research by Boccio, Fischer, Lundgren, Andrade et al. [28–32,36]. In this framework, the outcome of bond loss, derived from this experimental study and combined with other comparable data in literature, which are presented relative to the average crack width due to corrosion on the concrete's surface, shown in Figure 11, allowing for investigation of their correlation. In order to comment on the level of knowledge in the existing code

recommendations, the proposed predictive zones of Model Code 2010 on bond loss due to corrosion are compared to the experimental database.

Figure 11. Normalized bond strength due to corrosion as a function of average surface crack width on concrete, data from [16,17,23,36,39,41,44,45].

It is common knowledge that the development of surface cracking, which is inextricably linked to the corrosion of steel reinforcement, leads to degradation of bond strength between steel and concrete. As depicted in Figure 11, the presence of confinement contributes to bond behavior, delaying the reduction of the initial bond strength, while the amount of transverse reinforcement provides different levels of passive confinement in RC elements, which tend to follow a distinctive distribution.

To date, the recommendations of Model Code 2010 (Table 1) refer only to the presence or absence of stirrups (links), without mentioning the influence of their density on confinement and, subsequently, on bond loss. Since there is need to develop and/or enhance the existing codes, these findings can contribute toward this direction, by determining the magnitude of residual bond strength in more specific ways. In particular, the density of transverse reinforcement (stirrups density) should be included in the percentage of the proposed bond loss between steel and concrete.

On the basis of assessment of all experimental results, taking into account the proposed aggregation, as previously described in Table 3, a second regression analysis was performed, in terms of average crack width with respect to bond loss, for each level of confinement. This analysis provides an extension of a corresponding previous study by the authors [17], where the same approach was followed, adopting the same simplified exponential function, as described in Equation (7):

$$\frac{f_{bd,cor}}{f_{bd,non-cor}} = e^{-A \cdot c_w} \quad (7)$$

where A is a parameter that depends on the stirrups density in the RC element and c_w is the average crack width (in mm). However, herein more experimental data were used to increase the reliability level of the outcomes.

Currently, there are few experimental studies that correlate the normalized bond strength directly with the average crack width [16,17,23,36,39], including stirrups in the tested specimens. In the regression analysis, comparable experimental results of previous

scientific studies were included in order to extract predictive curves in terms of crack width. As in the case of previous analysis of bond loss (in relationship to the corrosion penetration), all recorded values of normalized bond strength greater than one were ignored, so as to take into account solely the degradation laws due to corrosion.

The outcomes of regression analysis for each group of confinement are depicted in Figure 12, along with the values of parameter A and the corresponding values of the R^2 coefficient, which are summarized in Table 5. The diagrams extracted (Figure 12) contain the experimental data, which were functionally separated with reference to the confinement level, in accordance with the proposed subgroups, as shown in Table 3.

Figure 12. Extracted exponential fitting curves of bond loss as a function of average crack width, for each confinement level group. (a) Unconfined specimens, (b) Slightly confined specimens, (c) Moderate confined specimens, (d) Narrow confined specimens.

Table 5. Parameters determined by regression analysis for the exponential predictive model.

	Group A Unconfined	Group B Slightly Confined	Group C Moderate Confined	Group D Narrow Confined
A	1.43	0.71	0.47	0.30
R^2 (%)	82	92	75	58

As demonstrated by the results of regression analysis (Table 5), the value of parameter A once again is decreased due to the increase of confinement level (number of stirrups); hence, it can be directly expressed via the volumetric percentage of transverse reinforcement. As a result, parameter A could be an effective index to estimate the ductile behavior of RC elements.

By analogy with the precedent regression analysis in terms of corrosion penetration and its results compared to Model Code 2010, Figure 8 refers to the bond loss due to corrosion in the presence or absence of stirrups, in terms of average crack width. Once again, in the case of absence of transverse reinforcement (no links), shown in Figure 12a, the recommendations of Model Code 2010 satisfactorily predict the residual bond strength of the corresponding tested specimens of group A (no stirrups), especially for crack width up to 1 mm. As in the case of corrosion penetration, in the case of presence of transverse reinforcement (with links), there is also a deviation between the predictive curves and the recommendations of Model Code 2010. More specifically, the boundaries proposed by Model Code 2010 tend to predict accurately the bond loss of group D (dense confinement) only, as shown in Figure 12d; nonetheless, group B (slight confinement) and group C (moderate confinement), shown in Figure 12b,c, demonstrate a gap compared to the recommendations of Model Code 2010, which overestimate the residual bond strength of existing RC structures. Eventually, the predictive boundaries for groups A, B and C need to be extended for greater levels of corrosion damage corresponding to an average crack width equal to 2.5 mm, according to the experimental data.

By extending the results of the regression analysis, this study proposes predictive zones of bond loss with respect to both aggressive environment and amount of transverse reinforcement, based on the upper and lower limits for each group of confinement level, presenting two different cases. More specifically, the first case (Figure 13) refers to modern designed structures, where the reinforcement detailing is more demanding, since moderate or narrow confinement is provided through dense stirrups spacing. On the other hand, the second case (Figure 14) refers to existing aging structures, where RC elements are slightly confined via a wide density of stirrups. Through this study, an attempt is made to estimate the level of existing knowledge on the correlation between the degraded bond strength due to corrosion and the surface crack width on concrete, discussing the recommendations of Model Code 2010 and its potential enhancement.

Figure 13. Proposed predictive zones of bond loss in relationship to crack width—in the case of modern designed structures.

Figure 14. Proposed predictive zones of bond loss in relationship to crack width—in the case of existing aging structures.

5. Conclusions

In the current manuscript, the impact of both corrosion damage of the main reinforcing steel bar and the amount of transverse reinforcement (stirrups density) on the degradation of bond strength of RC elements was studied, based on an ongoing extensive experimental program. The outcomes of the experimental campaign combined with data obtained from recent literature were compared with the recommendations of Model Code 2010. The following remarks can be made:

1. The amount of transverse reinforcement influences in great extent the rate and the magnitude of residual bond strength due to corrosion. The densification of stirrups in RC elements limits both the corrosion-induced cracking on the concrete surface and the reduction of bond strength between steel and concrete.
2. Given the fact that the recommendations of Model Code 2010 refer to the presence or absence of stirrups (links) as an on–off criterion, the present study indicates the need to update the current recommendations, taking into account the influence of stirrups density on both the surface cracking development and the bond loss between steel and concrete due to corrosion.
3. The large scatter of the existing experimental data from literature indicates the need to determine a common way of testing the bond mechanism. The geometry of tested specimens and the embedded length of the steel bar should be specified via standards. Nevertheless, the outcomes of the present study show that it is also crucial to examine the eccentric position of the main reinforcing bar and the amount of transverse reinforcement in order to simulate actual RC elements.
4. To enhance the current standards on the residual bond strength due to corrosion, taking into account the stirrups density, a discretization of confinement levels is proposed, according to experimental data, so as to represent different cases of stirrups density often occurring in actual RC elements, namely absence of confinement (no stirrups) and slightly, moderate and narrow confined in the presence of stirrups, as shown in Table 3.

5. Although the term of corrosion penetration does not fully represent the nonuniform corrosion damage of the embedded steel bars, yet to date it seems to be an appropriate assessment tool to correlate the residual bond strength with the corrosion level, since it does not affect significantly the bond performance.
6. The adopted exponential function, proposed in this manuscript, tends to be appropriate to predict the bond loss in relationship to both corrosion penetration depth and the average crack width. Drawing from the results of both regression analyses conducted, it is shown that the value of parameter A is reduced as the confinement level (number of stirrups) increases. Thus, it can be quantified directly with the volumetric ratio of transverse reinforcement.
7. On the basis of the abovementioned discretization depending on the confinement level, predictive zones of bond loss versus the corrosion penetration depth are proposed, in order to improve existing knowledge. Furthermore, the predictive boundaries for all cases need to be extended for greater levels of corrosion damage to improve the current recommendations.
8. Extending the results of the regression analysis, this study proposes predictive zones of bond loss with respect to both aggressive environment and the amount of transverse reinforcement, in terms of average crack width. As a result, the surface crack width on concrete is not yet an effective index to estimate the residual bond strength of RC elements, as there is a plethora of factors affecting the crack width. Hence, more experimental tests of comparable data are necessary.
9. The proposed predictive zones of the current study provide an appropriate assessment tool of the residual bond strength due to corrosion, in which the influence of transverse reinforcement is taken into account, thus amending the indicative values currently proposed by Model Code 2010.

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