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FINITE ELEMENT ANALYSIS OF BOND BEHAVIOR IN CORRODED REINFORCED CONCRETE BEAMS: STATE-OF-THE-ART

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The article conducts a comprehensive examination of various aspects related to rebar corrosion, encompassing the corrosion mechanism, its implications on design criteria, the modeling of bond interfaces under both corroded and non-corroded conditions, and the modeling of reinforced concrete (RC) beams affected by corrosion, employing both empirical and analytical methodologies. The initial stages of corrosion instigate a gradual transformation of rebar into rust. One notable consequence of reinforcement corrosion is the generation of expansive pressure, leading to concrete cracking, spalling, and detachment of the concrete cover. Additionally, it diminishes the effective cross-sectional area of the rebar, ultimately resulting in a decline in the concrete's bond strength and gradual structural deterioration. Ultimately, continuous corrosion can lead to a complete loss of bond between the concrete and rebar, representing the most severe form of damage attributable to corrosion. This poses a critical threat, particularly in cases where the beam functions as an unreinforced structure, potentially culminating in sudden structural failure. This paper primarily underscores the utilization of the Finite Element Method (FEM) for evaluating the impact of bond deterioration between concrete and reinforcement caused by corrosion. The paper effectively employs this technique to predict and analyze the structural damage in corroded RC beam specimens.

Keywords: corrosion, bond modeling, RC beams, finite element method

1 INTRODUCTION

Corrosion of concrete structures is a common problem occurring in RC structures since the steel reinforcing bar is susceptible to corrosion. Corrosion is the process of deterioration of metals or alloys which normally happens due to electrochemical reactions occurring in the acidic/alkaline environment. The reinforcement corrosion normally occurs in two phases. In the first phase, the corrosion process is initiated, wherein chloride ions (Cl^{-}) enter the concrete surface (through microcracks), propagate deep towards the reinforcing bar via connected pores, and get concentrated all over the rebars. In some cases, the concentration of these ions is localized, while in others these ions get concentrated all over the rebar surface. As the chloride ions concentration attains a threshold level, the passive layer gets ruptured, leading to the formulation of a corrosion cell over the rebar surface. Later, the second phase of propagation commences wherein the electrochemical reaction occurring leads to volume increase. Rust is generated as a by-product, with the progress of the propagation phase. It is arduous to evaluate the constituents of corrosion by-products. The volume occupied by rust is regarded up to 6.5 times that of the volume of reinforced steel consumed during the corrosion process, leading not only to a reduction in the effective area of steel as presented in Fig. 1[1], but also tensile stresses are exerted on the surrounding concrete.

Properties of concrete play a major role in resisting corrosion, especially the concrete in the region of cover. A dense concrete having a dense pore structure [2] will significantly resist the deteriorating fluids from reaching the reinforcement. Researchers, in recent times, have been focusing on the use of alternative binders as partial replacement to Portland cement to achieve durability as well as reduction in the environmental impact. Nevertheless, it should be noted that the rebar corrosion, generally, does not occur uniformly over the entire bar diameter, owing to varying rates of chloride ions ingress that could have resulted due to varying degradation of concrete cover. Although chloride ions penetrate along a single direction in real conditions, corrosion initiates from the outermost portion of the rebar and most of the time it does not undergo uniform corrosion. A comprehensive evaluation of the non-uniform propagation of corrosion is reported in several studies[3]–[5]. The corrosion of reinforcement could also lead to loss of its effective area in cross-section along with a gradual bond degradation around concrete and the rebar. Further, being, both circumstances could endanger the safety of the structure and also dreadfully affect the serviceability of the structure, besides vitiating system durability.

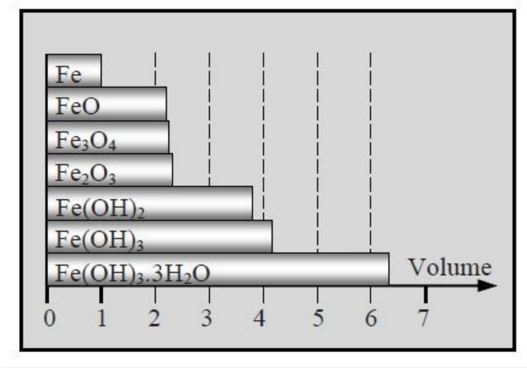

Fig. 1. Oxidation states of Iron[1]

There are many environmental and material factors influencing the corrosion performance of reinforced concrete. Researchers have studied experimentally as well as analytically the influence of corrosion on the performance of reinforced concrete structural members. This paper aims to provide a comprehensive state-of-the-art review of factors influencing bond deterioration and resulting deterioration of bond and mechanical properties. Corrosion is influenced by many parameters which is very difficult to stimulate experimentally. However, these can be modelled using various FEM methods.

2 MECHANISM OF CORROSION

The mechanism of reinforcement corrosion is similar to the electrochemical reactions that take place within a flash battery[6]. The top layer of the rebar behaves as an electrode composed of cathodes and anodes that are interlinked over the rebar surface electronically. The associated cathodic and anodic reactions occur throughout this rebar surface[7]. To support this, the aqueous medium of pore water inside the concrete facilitates the reaction as it consists of some complex electrolytes. The mechanism of corrosion in RC beams can be explained by association with the formation of a cell as represented in the Fig. 2.

The corrosion cell consists of the cathode, anode, and electrolyte. The reactions that eventuate during the process of corrosion are widely described as the Half-Cell Reactions and involve the oxidation of metal and reduction of oxygen. During the corrosion process, the reactions that occur at the anode region is the oxidation of the metal and the electrons are supplied by the cathode.

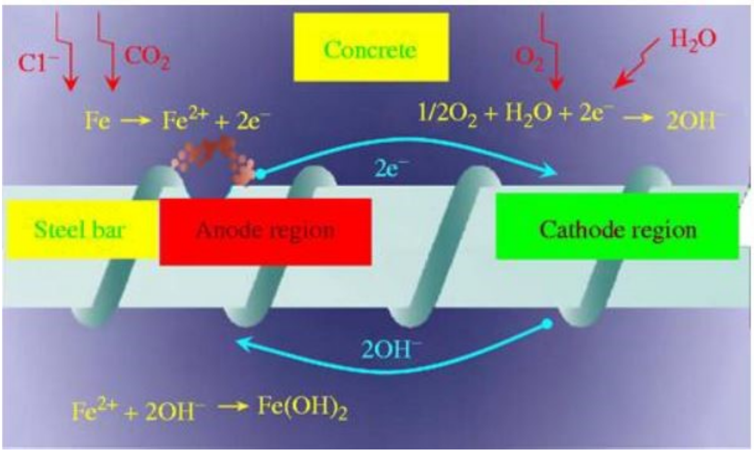

Fig. 2. Electrochemical process of Rebar Corrosion in Concrete [8]

$$Fe \rightarrow Fe^+ + e^- \quad (1)$$

Reduction of the hydrogen ions takes place during the cathode process in an acidic environment (pH<7)

$$2H^+ + 2e^- \rightarrow H_2 \quad (2)$$

The reaction that takes place at the cathode is the dissolved oxygen reduction in an alkaline environment or neutral (pH>=7)

$$O_2 + 2H_2O + 4e^- \rightarrow 4OH^- \quad (3)$$

Equations 1, 2, and 3 describe the mechanism of corrosion that generally occurs around rebar in acidic and alkaline environments. It is to be noted that the mechanism is supported by various environmental factors such as temperature and moisture conditions.

3 METHODOLOGY

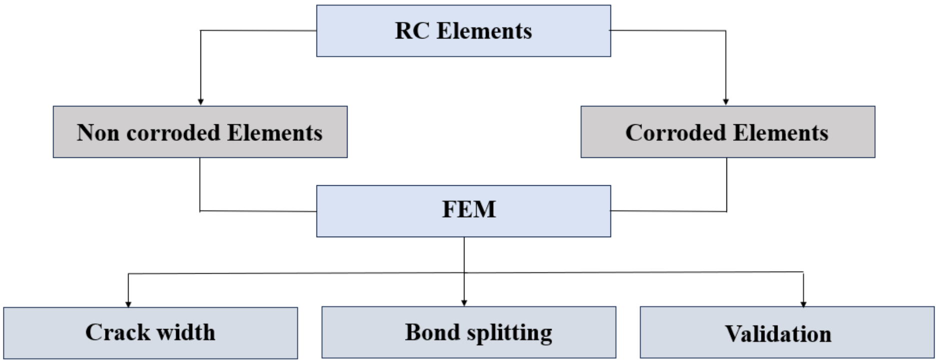

Fig. 3. Flow chart of Methodology

Initiate the process by conducting a thorough assessment of Finite Element Method (FEM) modeling for Reinforced Concrete (RC) Elements. Subsequently, categorize the RC elements into two groups: Non-Corroded and Corroded elements. Proceed to perform an extensive literature review, focusing on the Finite Element Method applications in both Non-Corroded and Corroded RC elements. Following the literature review, execute a Parametric study that investigates factors such as crack width and bond splitting. Additionally, validate the FEM models to assess their performance using various materials. Compare the FEM model predictions with experimental results. Conclude the methodology by summarizing the findings and deriving meaningful conclusions based on the results obtained from the Parametric study and validation process as represented in the Fig. 3.

4 IMPACT OF CORROSION ON THE DESIGN OF RC STRUCTURES

Steel reinforcement in any form, be it used as tensile or shear irrespective of the structural loads, is susceptible to corrosion. Many times, stirrups corrosion and the resulting damage to the shear capacity of RC structural elements, is indeed an underestimated area of study under steel reinforcement corrosion [9]. Stirrup corrosion results in spalling and subsequent delamination of concrete. The corrosion-induced cracking associated with reinforced concrete behavior is studied by many researchers. However, literature on corrosion proofing and examining the concrete condition for stirrup corrosion is laborious work, hence very few studies are reported. The cracks developed by stirrup corrosion proceed transversely to the main bars, but the width of these cracks is, however, smaller. To Add to this, the numbers are larger than that of the cracks developed due to corrosion of the longitudinal bar. Stirrup welding with longitudinal reinforcement would contribute to the development of a cathode condition for longitudinal reinforcement corrosion.

Laboratory test on corrosion is performed using accelerated corrosion technique. Pandit P (2019) conducted experimental research on RC beams, cast using Ordinary Portland Cement (OPC) and Portland Pozzolana Cement (PPC), to assess the flexural behavior of beams [10]. A maximum of 10% corrosion was induced using the Accelerated Corrosion Technique (ACT). Using the Applied Corrosion Monitoring (ACM) device the rate of corrosion was measured, and it was noticed that the corrosion rate in OPC beams was 15% higher than in PPC beams[11]. All the samples were tested as cantilever beams with an end bearing length of 400 mm fastened in a rolled steel joist testing frame weighing 15 tonnes. The loading point and the span remained unaltered for all the beams. A concentrated load was applied at the free end of the beam. The deflection often rises with the increase in corrosion rate until 10%. However, the load-carrying capacity of both OPC and PPC beams had been decreased in comparison with the control beam specimens, owing to the variation in the elastic modulus of steel. As a consequence, the corroded beams failed at a lower load experiencing even more deflection than the controlled beams. However, a non-linear decrease in the rigidity of the beams was observed.

According to Val et al. (2009), the reduction in strength of reinforced concrete due to the increasing rate of corrosion is a well-established fact. However, assessing the contribution of various factors affecting strength i.e., materials, loads and corrosion rate is a crucial task. Therefore, a probabilistic model was developed to correlate the loss of strength and the corrosion rate[12].

Wang et al. (2012) also recorded similar observations concerning continuous monitoring of the structural elements undergoing pitting and uniform corrosion [13].

According to Bhargava et al. (2008), the mechanism of bond development between concrete constituents and reinforcement is still a strong area of research, as the bonding is influenced by a wide range of factors influencing concrete properties. The bond capacity of concrete is affected by both the rebar diameter and crushing strength of concrete. The outcomes of the previous works are contradictory, concerning the influence on bond capacity owing to corrosion. Nevertheless, the considerations included in the study are of sizeable interest in the quality of the research 16. The empirical model established to correlate bond capacity and reinforcement corrosion acknowledges that reinforcement corrosion deteriorates the bond capacity of RC structures [14].

5 BOND MODELLING WITHOUT CORROSION

Analytical tools have been an effective way to predict the behavior of structural performance under various factors that are difficult to stimulate experimentally. Few researchers have tried to stimulate the bond performance in RC structures, incorporating various influencing factors which are experimental stimulation is difficult or many times unpracticable. Table 1 provides consolidated data on bond studies carried out using FE modelling. The simulation of the bond was attempted by Henriques et al. (2013) using the cohesive contact behavior (The uncoupled traction-separation equation approximated the bond-slip curve). For rebars, as well as for concrete, solid 3-D element type, C3D8R was used in Abaqus CAE. Two tests were performed to validate the results. Firstly, the hooked bars integrated into concrete blocks were subjected to pull-out tests. Secondly, an RC beam, simply supported on either end, was loaded with a point load at the center until failure [15].

Ogura et al. (2008) performed the 2-D planar Finite Element (FE) analysis to examine the influence of the location of main rebars, compressive strength of the concrete, and transverse reinforcement on the splitting failure of the bond. Outcomes of the FE simulation on samples of lap splice and pull-out tests revealed that the splitting strength of the bond measured in a laboratory test is nearly equal to the peak mean radial stress obtained from FE analysis. Additionally, it had been observed that the compressive strength of the concrete will regulate bond capacity in case of a larger amount of transverse reinforcement. However, the effect of the residual tensile load capacity of concrete in the splice area cannot be disregarded [16].

Dehestani et al. (2015) studied the bond-slip behavior, with a basic embedded steel rebar element with the application of an equivalent strain in the FE analysis. Consequently, effective reinforcement stiffness dropped considerably for subsequent analysis of the model, reflecting the advent of the corrosion process. The model was then validated utilizing Abaqus CAE (a non-linear FE software package) with the laboratory simulation studies. The software demonstrated its ability to reflect the bonding effect with the use of embedded aspects in the study of RC structures. A comprehensive parametric study was performed to assess the effect of the subsequent parameters like materials adopted for concrete and steel, the diameter of the rebar, reinforcement ratio, and the containment conditions[17].

Table 1. FE modeling of bond characteristics

Reference	Type of Element	Grade of Concrete (N/mm^2)	Grade of Steel (N/mm^2)	Failure Load (kN)	Crack Pattern	Bond Slip	Effectiveness of the Numerical model Concerning Experimental
[18]	Beam	53	Fe 415	40	Micro cracks observed at the interface of steel and Concrete		Fairly good
[17]	Slab	40	Fe450 to 600	80		The percentage of reinforcement ratio does not affect bond slip	Fairly good
[16]	Beam	64	327			The compressive strength of Concrete will govern bond strength	Fairly good
[19]	Column			50		Cyclic bar slip reversal technique used	Fairly good
[20]	Cylinder	30	415			If bond strength decreases increase the bond-slip value	Fairly good
[21]	Beam 150×150×550 mm	30 to 60	Fe 400			The bond between coated rebar and concrete can be enhanced by increasing concrete strength or increasing the cover concrete's thickness. The increase in the rebar diameter will reduce the bond strength between coated rebar and concrete	The slip corresponding to the ultimate load obtained by ANSYS finite element simulation is less than 20% different from those measured results of the beam test
[22]	Cylinder and Cubes	30 to 50	GFRP Plate of 695			User-defined bon slip model developed	implemented into commercial FE software and used for structural analysis. Error (0-20%)

Murcia-Delso et al. (2015) introduced a novel interface design model that simulates the bond-slip behavior of rebars. The loss of bond caused by cyclic slip reversals, concrete splitting, and steel yield strength are accounted, for by adopting a semi-empirical law. The analysis results indicated that the model can also be used precisely to simulate splitting bond failure and to assess the bond stress-slip performance of reinforced bars in confined concrete[23].

Wu et al. (2013) presented a bond-slip model by making use of a single continuous equation which felicitates numerical investigations. This equation of the model does not distinguish between plain or confined concrete and pull-out or splitting failure. The numerical outcomes indicated that the proposed model surpassed current models, in the precise way it predicted[24].

The bond behavior around the rebar-concrete interface in RC tie members was studied by Ziari et al. (2014) using the non-linear FE technique. Three distinct FE bond models, namely diagonal link element, perfect bond, and bond zone were developed. The reliability of these bond modelling types is assessed against laboratory results obtained from two other researchers[25].

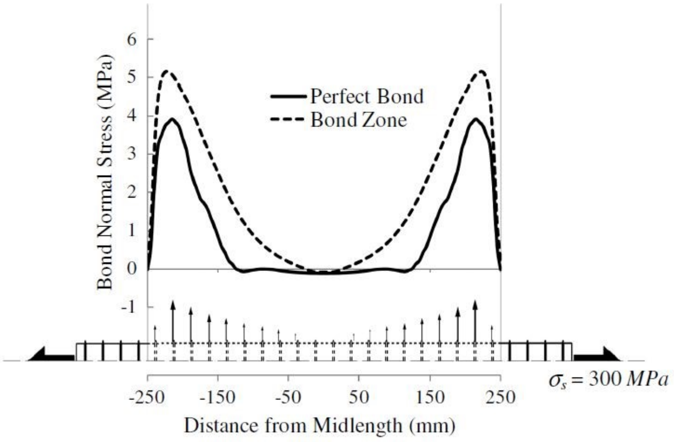

Fig. 4. Comparison of normal stresses of bond for perfect bond and bond-zone models[26]

Similar to the test data, the first crack was developed at a stress of around 300MPa as shown in Fig. 4. Nevertheless, a further crack was developed for a perfect bond model at a stress of around 390MPa, which was not in compliance with laboratory results. In the perfect bond model, this was an indicator of over-estimation of bond shear stresses. Further, the findings of the bond zone model did not demonstrate the presence of a second crack. Moreover, FE outcomes in the bond zone model were closely related to the laboratory results compared to the perfect bond model. Consequently, the bond zone model was adopted to obtain the desired crack width and crack patterns. Based on the findings of FE analysis, a multi-linear pattern of curve was presented to estimate the bond shear stress range as shown in Fig. 5.

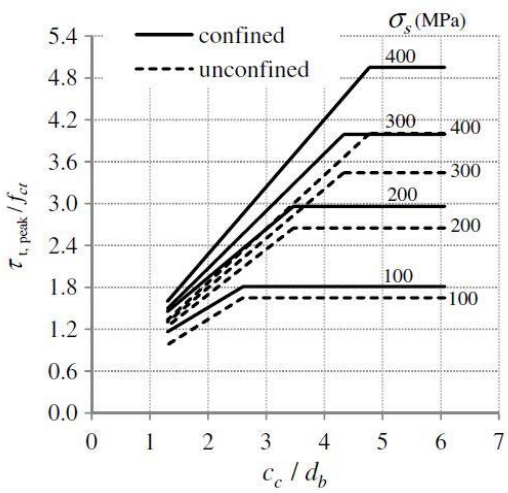

Fig. 5. Idealized maximum bond shear stress of confined and un-confined concrete[26]

The bond zone model is effective in measuring both normal stress and bond shear. Using the bond zone model, the mean split tensile stresses and the maximum bond shear stresses induced due to normal bond stress are evaluated by parametric FE analyses.

A FE bond element including the steel strain effect was introduced by Santos et al. (2015) with a bond stress-slip relation. The conceptual element embodies a four-noded orthotropic plane-stress structure and revised constitutive material provisions. The outcomes of the validation demonstrate the need for incorporating this element to attain better results. The bond element and the suggested reduction function were also emphasized as imperative in the subsequent issues: tension stiffening, spacing between the cracks, minimum reinforcement, deflection, ductility, and rotational capacity of cracked RC structures[26].

6 BOND MODELLING WITH CORROSION

FE provided a versatile platform to study the bond characteristics of reinforced concrete structures with induced corrosion and its influence on bond characteristics. Table 2 provides consolidated data on bond studies carried out using FE modelling extracted from recent literature. The preliminary FEM research conducted by Bossio et al. (2015) on a cylindrical model, encompassed with only one rebar to reflect the pitting or the uniform corrosion of the reinforcement. Additionally, another model has been suggested for the mechanical investigation of stresses in the region of concrete enveloping over bars. Furthermore, a well-refined model was developed to analyze the non-linear stress distribution and propagation of cracks within the concrete, as rebar triggers to deteriorate due to corrosion. The findings reported an agreeable correlation between the reduction in the steel area and the progress of cracking. Eventually, there was a reasonable agreement between numerical observations and the literature study results[28].

Table 2. FE modeling of bond characteristics for corroded RC elements

Reference	Type of Specimen	Grade of Concrete	Grade of Steel	Failure Load (kN)	Crack Pattern	Bond Slip	Effectiveness of the Numerical model
[29]	Prism 100×100×400 mm	50	CFRP			A bond-slip model was developed FRP-to-concrete interface	Fairly good agreement
[28]	Cylinder	M20 to 30	Fe 500		The crack develops in a radial direction		Fairly good agreement
[30]	Cylinder	40	500			For large corrosion penetration, the numerical model will give non-conservative results	Fairly good agreement without corrosion
[31]	Beam	40 to 50	Fe 415				Fairly good agreement
[32]	Beam	40	Fe 415		Crack width varies concerning time		Fairly good agreement
[33]	Beam	15 to 60	Fe 415		A new theoretical model developed for non-uniform corrosion		Fairly good agreement
[27]	Beam		BFRP			Presence of BFRP improves good bonding	

The one-dimensional (1-D) analytical model was introduced by Lundgren et al. (2012) to reflect the analytical relation between bond stress-slip behaviour and bond stress of corroded rebars. A bond stress-slip model has been provided including the parameters attributed to shift in the failure manner to represent bond deterioration. Additionally, the analysis showed the computation of the required anchorage length using a differential equation for a 1-D bond stress slip [30].

Bhargava et al. (2006) proposed the bond as well as bond disintegration model utilizing a contact interaction mechanism for pull-out study in the Abaqus CAE environment. In the instance of concrete cracking, the strain perpendicular to the interface of concrete and rebar, and the friction between them is alleviated. In pull-out experiments, this significant reduction in friction and pressure was evaluated through various levels of corrosion. The outcome was predicated on the correlation that specifies deterioration of bond, contact pressure, and friction concerning loss of mass due to corrosion[31].

Jnaid (2014) investigated the bond deterioration due to corrosion and was evaluated by analyzing the residual ultimate bending strength of unbonded beams. The ultimate stress in the pre-stressed concrete measured by the 'depth of neutral axis' method has been revised. An FE model was generated using ANSYS application software and validated with multiple samples (109 beams) of experimental data that included several parameters like loading type, shear length to the effective depth (a/d) quotient, and length of un-bonded region. The experimental data and the FE model were used to provide an analytical model to estimate the remaining flexural power. The findings indicated that the un-bonded length and the reinforcement ratio influenced considerably the residual flexural strength of the beams with un-bonded rebars[34].

Lee et al. (2002) experimental and FE simulations were conducted on accelerated corrosion specimens for pull-out experiments. It was reported that with the rise in corrosion degree, bond capacity, and stiffness drop significantly. The research suggests numerical equations for evaluating the bond capacity and stiffness according to the corrosion rate[35].

Bhargava et al. (2006) studied the weight loss and time required to trigger cracking of concrete. Concluded that the time taken to initiate a crack in concrete was strongly influenced by the tensile modulus of steel, the tensile capacity of concrete cover, corrosion products, and annual median corrosion rate[31].

The analytical model was developed by Bhargava et al. (2007) to measure the residual bond capacity of concrete. Further, the model accounted for several variables like adhesion and friction between corroded rebar and the cracked concrete, expansive pressure around the interface due to accumulation of rust, and simulation of cracked concrete for tensile behavior[36].

A basic empirical model was introduced by Bhargava et al. (2008) for the computation of residual bond capacity based on corrosion in the concrete reinforcement without stirrups. The current research will be very effective in estimating the design life of RC components of the structure owing to the gradual deterioration of bond capacity due to rebar corrosion. This research is very useful in studies, involving flexural and residual load capacity evaluation, for different configurations of structural components. The research also acts as a basis for further progress in studies involving stirrups[14].

The research was conducted by Berto et al. (2008) to introduce two novel techniques for the simulation of the bond: damage and friction types. The entire structure of the stress-slip curve was altered in the damage type, concerning the degree of corrosion. In friction type, the degradation of the bond was reflected through modification of the overall bond capacity (τmax) and the gradient of the beginning linear portion of the bond stress-slip curve. Eventually, laboratory results obtained for corroded samples of the beam were simulated numerically[37].

The analytical model was proposed by Chen et al. (2015) to measure the residual bond capacity and to estimate the crack propagation in the concrete cover. For concrete protection, the thick-walled cylinder model was incorporated concerning residual tensile strength, decreased tensile rigidity, and anisotropic behaviour. The governing equation formulated comprising confinement, corrosion pressure, corrosion rate, and adhesion was used to measure the radial burst resistance in the bond layer and the width of the crack over the cover region of concrete. Consequently, the proposed research model has been effectively validated utilizing the relevant experimental results[38].

A constitutive law was established by Richard et al. (2010) to model the interaction between steel and concrete, combining the effects of corrosion. The investigation presented a 3-D system relying on the mechanics of continuum damage. To illustrate the productivity of the model, a pull-out test was simulated on corroded and uncorroded samples. The study involved a qualitative distinction between the investigational crack path as well as the damage pattern due to corrosion[39].

7 CORRODED BEAM MODELING

The non-linear FE modeling in the Abaqus CAE software package initiated by German et al. (2015) incorporates a 2-D FE model of a corroded beam sample. The by-product of corrosion i.e., rust was considered as an interface part in the model. The 2-D model is focused on simulating the effect of rebar corrosion on the concrete cover. In the cross-section analysis, non-linear computations of implicit and explicit algorithms were employed and compared. Eventually, a 3-D FE simulation of the beam prone to corrosion and static loading was studied[40]. The order of applying the displacement and corrosion load was altered. In the end, the concrete deterioration was the same irrespective of how the load was applied. The steel, concrete, and rust parameters used are presented in the table 3 and 4 below.

Table 3. Material properties for 3-D model[40]

Material	Parameter	Value
Concrete	Elastic Modulus, E (MPa)	20000
	Poisson's ratio, υ	0.2
	Peak stress at compression, f_c (MPa)	38.3
	Peak strain at compression, ε_c	0.2%
	Tension yield stress, ft (MPa)	2
	Fracture Energy, G_f (N-m/m^2)	0.1
Steel	Elastic Modulus, E (MPa)	200000
	Poisson's ratio, υ	0.3
	Yield stress, σ_e (MPa)	350
	Plastic Tangent stiffness, E_T (MPa)	3295

Table 4. Dependence of the Rust interface properties on corrosion rate[40]

Bond Strength (MPa)	Bond Stiffness (N/mm^3)	$\delta_m^f - \delta_m^0$	Corrosion Rate (%)
25	1000	0.5	0
30	500	-	1

Bond Strength (MPa)	Bond Stiffness (N/mm^3)	$\delta_m{}^f - \delta_m{}^0$	Corrosion Rate (%)
25	-	-	2
21	500	0.1	4
8	88	1.5	6
3	35	1.5	8

Han et al. (2014) developed the flexural capability evaluation model for corroded RC sections. The method was built on the inference of rebar-concrete bond interface models which rely on the theory of thick-walled cylinders. The theory of median expansive pressure was integrated with the non-linear model for flexural behavior study and onto the model for bond capacity to constitute this novel model. The established model was validated with 59 corroded RC flexural samples drawn out of four distinct research models and has shown a quite accurate forecast of flexural strength depletion[41].

Coronelli et al. (2004) generated a 2-D non-linear FE model to research the effect of corrosion on beam efficiency. Several damage criteria have been incorporated into the existing model, which includes bond loss, crushing and cracking, rebar yield, and the evaluation of the specific degree of safety. The effect of corrosion in the non-linear FE analysis was simulated by altering the characteristics of the components, like the rebar area, and then by adopting the applicable material rules and hence the bond between the rebar and concrete. The spalling and cracking of the compressed concrete are represented by decreasing the capacity of the concrete elements around the cover. The adoption of a particular model for bond degradation was ascertained to be of critical significance in assessing the residual structural rigidity of the elements[42].

The RC beams subjected to natural corrosion were examined by Almassri et al. (2015) for almost 25 years, restored using Near Surface Mounted (NSM) Carbon Fiber Reinforced Polymer (CFRP) bars, and explored the failure mode of beam focused on laboratory findings and outcomes of the numerical simulation. The study results were obtained for experimental and numerical simulation of a 2-D FE model adopting the FEMIX programming tool. The numerical FE model results of the FEMIX code were then validated satisfactorily with the laboratory outcomes, except modified corroded beam that required a 3-D FE model employing the Abaqus CAE commercial software package. Regardless of non-classical collapse mode and concrete cover dissociation, owing to damage caused by corrosion, the NSM approach enhanced the total capacity (ultimate and yield capacities) for the control and corroded beams. 3-D FEM analyses using Abaqus CAE were capable to predict both ultimate deflection and load-bearing capacity, as a virtue of corrosion when the crack plane caused by corrosion had been incorporated into the model[43].

An analytical model was developed by Maaddawy et al. (2005) to evaluate the response of both controlled and corroded beam specimens. Throughout the study, the deflection of the RC beam was assessed based on elongation amongst the flexural cracks of rebars instead of the beam curvature. Based on the published research, a novel bond-slip model was formulated to quantify bond capacity loss because of corrosion. The reliability of the model has been validated and the findings indicated that the model is effective in plotting the graphs for load vs. deflection for both corroded and controlled beam samples. The bond strength model developed to evaluate the maximum bond capacity of the corroded RC Beam is as given by the following equations[44].

$$\tau_{conc} = \left(0.55 + 0.24\frac{c_c}{d_b}\right)\sqrt{f_c{}'} \tag{4}$$

$$\tau_{st} = 0.191\frac{A_t f_{yt}}{s_s d_b} \tag{5}$$

$$\tau_{max,v} = \left(0.55 + 0.24\frac{c_c}{d_b}\right)\sqrt{f_c{}'} + 0.191\frac{A_t f_{yt}}{s_s d_b} \tag{6}$$

$$R = A_1 + A_2 m_l \tag{7}$$

$$\tau_{max,c} = R\left(0.55 + 0.24\frac{c_c}{d_b}\right)\sqrt{f_c{}'} + 0.191\frac{A_t f_{yt}}{s_s d_b} \tag{8}$$

Kallias et al. (2010) analyzed the non-linear 2-D FE model to assess the structural behavior of corroded beams in the DIANA application software tool. The validation of the FE model concerning load deflection properties was carried out in reasonable agreement with laboratory trials conducted by other researchers. Concerning altering the structural configuration of concrete and rebar, the influence of corrosion was incorporated in the FE study by altering the fundamental content relation of the concrete, bond materials, and rebars. The influence on both the ultimate limit and the serviceability conditions was studied. When tensile reinforcing bars are firmly anchored on both ends, increasing the bond loss leads to larger crack width and spacing. However, the ultimate strength of the beam specimen was not influenced. Further, noted that minimizing the damage to concrete cover in the compression zone tends for an over reckoning of the structural response or behavior during both the ultimate limit states or the serviceability conditions[45].

The analytical and FE modeling was carried out by Hanjari et al. (2011) to analyze residual strength and the responses of corroded RC components. The concept was worked out by altering the characteristics and configuration of concrete and corroded rebars by changing bond-slip characteristics, reducing rebar surface area and elastic modulus, and adjusting the response of concrete due to corrosion cracks[46].

Biondini et al. (2014) introduced a non-linear 3-D FE element for analyzing the corroded RC beams that reflect both material and geometric nonlinearity. The FE method of modeling addressed both uniform and localized corrosion considering delamination and concrete cover spalling, splitting cracks of concrete as well as the loss in rebar area. However, bond losses were not incorporated into this analysis. The results obtained from the numerical investigation of FE models observed a strong validation with the experimental findings of corroded beam specimens. The FE analyses for both the 3-D RC arch bridge and the statically indeterminate RC beam subjected to various degradation conditions and the corrosion rates were conducted as part of the implementation of the suggested formulation[47]. The ultimate strain, εsu of the corroded rebar has been related to the damage index δs = δs(δ) using the following relation.

$$\varepsilon_{su} = \begin{cases} \varepsilon_{su0}, & 0 \le \delta_s < 0.016 \\ 0.1521\delta_s^{-0.4583}\varepsilon_{su0}, & 0.016 \ge \delta_s \le 1 \end{cases} \tag{9}$$

Also, the amount of rebar damage needed for the initiation of crack δs0 was evaluated by the following equation.

$$\delta_{s0} = 1 - \left[1 - \frac{R}{D_o}\left(7.53 + 9.32\frac{c_o}{D_o}\right) \times 10^{-3}\right]^2 \tag{10}$$

Potisuk et al. (2011) researched beams with tensile-influenced behavior. FE model was created to analyze the influence of corrosion degradation factors like uniform and pitting corrosion of stirrups, concrete cover spalling, and loss of bond between concrete and stirrups, over the structural response of beam under corrosion. FE analyses were carried out to ascertain both individual and collective losses. FE and laboratory findings demonstrated strong validation of the residual strength of the corroded beams[48].

8 DISCUSSIONS

Given that corrosion is a complex and unpredictable phenomenon, extrapolating these results will necessitate further research, extensive numerical analysis, and rigorous validation. To evaluate the impact of random or spontaneous corrosion on flexural behavior, additional investigation is crucial since many studies have not accounted for localized or non-uniform corrosion.

The development of a comprehensive model that encompasses all potential corrosion effects, such as deflection, crack density, spacing, rotational capacity, minimum reinforcing steel requirements, tension stiffening, and ductility of corroded reinforced concrete structural elements, is essential to predict the performance reduction function and bond element. It's worth noting that the rebars are installed using a near-surface mounted technique, which enhances both the ultimate and yield capacities for both the control and corroded beams. The simplified damage plasticity model has been determined to be effective in simulating the impact of corrosion on reinforced concrete beams Top of Form

Due to the reduced chloride migration in PPC beams compared to OPC beams, PPC beams exhibited a 15% greater resistance to corrosion than OPC beams. In the study, the Accelerated Corrosion Technique (ACT) effectively halted concrete spalling in RC beams when the corrosion rate reached 10%. However, corrosion rates exceeding 10% were not taken into consideration, as the initiation of concrete spalling in the protective cover zone could significantly impact the beam's structural capacity.

Two essential empirical models, derived from the results of experimental pull-out and flexural tests, are introduced to evaluate the residual bond strength of concrete components, even in cases where stirrups are absent. The analytical assessments focused on flexural behavior are observed to be more cautious in their predictions when compared to the model based on the pull-out test. These results indicate that the proposed empirical models may possess the capacity to reliably forecast the residual bond strength of a concrete component that has been influenced by rebar corrosion.

Furthermore, the research highlights that rebar corrosion has a substantial adverse impact on the flexural strength of concrete beams with deteriorated bond conditions.

9 CONCLUSION

1. FE modeling offers a flexible framework for simulating rebar corrosion, shear reinforcement corrosion, and the subsequent effects on the degradation of mechanical properties.
2. Many studies are focused mostly on 2-D modelling. However, most of the FE and analytical modelling techniques that are now available are for shear or flexural critical specimens of corroded beams.
3. The experimental investigation conducted to verify the Finite Element Model (FEM) yielded only satisfactory results. However, it's important to note that a significant aspect, namely the modeling of the interaction between the rebars and the concrete, was not explicitly addressed throughout the entire study.
4. Recent studies have not adequately addressed the prolonged loading conditions that structural components often experience in real-life structures.

5. The proposed finite element model, which employs a simplified damage plasticity model, has been determined to be effective in simulating the impact of corrosion on reinforced concrete beams.

9.1 Scope for Future Study

1. A model could be proposed to provide a basis for numerical evaluation of the corroded RC beams, considering the impact of pure torsion or dynamic loading. Also, FE models could be developed for Pre-Stressed Concrete beams subjected to stirrup corrosion.
2. Further simulations on the structural behavior of the corroded rebars and concrete, considering finite element approaches and incorporating the bond mechanism, can be performed to help assess the torsional moment potentials of corroded structural elements.
3. Even further experimentation and numerical evaluation, are, however, necessary to determine the influence of the location of corroded torsional rebars upon torsion.
4. A 3-D model could be introduced, to simulate out-of-the-plane damage at the rebar-concrete interface, induced by expansive pressure of rust due to corrosion.
5. Nonetheless, no study has been reported on predicting the degradation of binding coercion, fatigue, and tensile strength at the rebar-concrete interface zone. Also, its impact on the bond behavior owing to combined exertion of corrosion and sustained loading.

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11 NOMENCLATURE

τ_{conc}	=	contribution of concrete to bond strength
c_c	=	Lesser of one-half rebar spacing and clear cover to concrete
d_b	=	diameter of the tensile reinforcing bar
f_c'	=	concrete compressive strength
τ_{st}	=	contribution of stirrups to bond strength
A_t	=	cross-section area of stirrup
f_{yt}	=	stirrup yield strength
s_s	=	spacing between stirrups
$\tau_{max,v}$	=	maximum bond stress of the uncorroded part of the beam
R	=	the factor that characterizes the change in bond strength caused by corrosion
A_1, A_2	=	variables that depend on the current density level
m_l	=	Percent mass loss of steel due to corrosion
$\tau_{max,c}$	=	maximum bond stress of corroded part of the beam
ε_{su}	=	an ultimate strain of corroded rebar
ε_{su0}	=	an ultimate strain of undamaged rebar
δ_s	=	damage index
δ_{s0}	=	the extent of damage to steel required for the onset of crack.

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