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Review

Review study towards corrosion mechanism and its impact on the durability of concrete structures

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Abstract: This paper imparts a review study on the causes and factors responsible for corrosion, its initiation and propagation mechanism inside the structures, leading to a better discernment of the problem associated with the durability of the existing structures. This study employs all necessary information related to the corrosion activity, fostering the researchers towards explicating some productive outcome to enhance the durability characteristics as well as the service life of reinforced concrete structures. Different techniques like, Impressed current technique (normally in a range of 1 mA/cm^2 to 4 mA/cm^2) has been adopted by several authors to induce corrosion artificially to better correlate the result with the natural form of corrosion in the structures. This paper particularly emphasizes on residual flexure and shear capacity of reinforced concrete sections undergoing corrosion mechanism, the effect of which leads to a reduction in strength of up to 50%–60%. Several empirical relationships for the prediction of residual load capacity (flexure and shear) of a reinforced-concrete members and relationships for the surface crack width (lying in a range of 0.05 mm to 15 mm) and weight/mass loss (average loss of 15%–30%) were established based on the data obtained by various experimental observations.

Keywords: reinforced concrete; durability; permeability; carbonation; corrosion density; service life prediction; residual strength

1. Introduction

Concrete has been proved to be a vital element in the construction field since majority of the construction industry involves design of reinforced concrete structures to accommodate with the convenience of every sections of human habitants and the materials of which is easily and widely available in the market. Reinforced concrete members are made up of three main ingredients: coarse aggregate, fine aggregate, and cement. Corrosion is similar to cancer for the reinforced concrete structure because it degrades reinforcement which affects its strength, life span [1,2]. Because of corrosion, the rust product formed in the surrounding zone of reinforcement occupies a larger volume leading to a development of tension cracks in that zone leaving behind a reason for sudden collapse or menace to the structure [3]. Different type of concretes need different type of precaution depending upon the climatic and material factors. As time and load increase, durability of structure decreases because of the action of salts, climatic condition, corrosion etc. [4].

Corrosion risk in reinforcement depends upon alkalinity of concrete (main compound is chloride) which is directly proportional to the severe causes of corrosion (high, moderate, low intensity corrosion depending upon risk) also on the factor of mixture whether it is carbonated or un-carbonated [5,6].

1.1. Factors responsible for Corrosion

There are several factors like chemicals, salts, cracks, pollution, cover, quality of material, mixing composition and on-site usage of material and its handling etc. which promotes corrosion activity [1]. Due to the improper compaction of concrete, mixing, water cement ratio, hydration of cement, and different additional materials, creates porosity which assists chemicals to infiltrate in concrete and decrease pH value, which leads to decrease in the durability of structure as time duration and effective load increase [4].

Chloride is a salt compound which is highly reactive in nature and causes decrease in alkalinity (decrease in pH), as concrete's pH value decreases below 9.5 the probability of corrosion reaction is effectively increased. After chloride salt ingress in concrete, most of the particles reacts with the cement and form calcium chloro-aluminates and calcium chloro-ferrites but some of the particles remain unreacted and chloride-hydroxides ratio decreases which leads to decrease in the pH of concrete [7].

Sulphate is also a salt compound similar to chloride compound. It's working, basic phenomena and effect on concrete are same but ionic and compound reactions are different than chloride reaction. Both salts decrease the concrete alkalinity and provides favorable environment for corrosion reaction.

Carbonation is a process in which atmospheric carbon dioxide (CO_2) reacts with un-reacted particles of cement or salts and decrease the alkalinity of concrete which further damages the protective layer of steel. Ingression of Carbon dioxide within the concrete is either due to permeability of concrete or carbon dioxide trapped in concrete because of hydration reaction in cement [8].

Alkalinity is defined as the basic nature of the concrete, greater the value of pH (>7) grater will be the alkalinity. Below pH level 7 acidic environment develops which enhances the rate of corrosion reaction [4]. In case pH value drops below 9.5, the probability of further reactions increases [4].

Normally concrete is alkali in nature having pH 11.5 to 13.5 at the time of construction, due to the presence of salts (chloride and sulphate) or carbonation reaction alkalinity gets decreased [7]. **Permeability** is the process in which water flows within the concrete system by the means of pores and cracks. Environment constitutes various elements like salts, moisture, and pollutants etc. which percolate through voids from surface to reinforcement and participate in chemical reactions [9]. The more voids within the concrete system increases the rate of permeability. Less permeable concrete slows down the percolation of elements within the concrete from environment [10].

Cover is also plays an important factor which directly relates with the corrosion process. Nominal cover is supposed to be maintained as prescribed by IS 456 for different exposure conditions. An appropriate cover certainly prevents the reinforcement from exposure. It should be adopted meticulously keeping in view of the climatic and geographical effects, type of soil and salts present in soil etc.

Cracks are the major factor for the enhancement of corrosion rate. Cracks can be developed due to different factors like internal/external pressure or loading criteria, curing, material, mixing, environmental effects, improper compaction etc. It accelerates corrosion process by increasing permeability.

There are many other factors which are also responsible for the corrosion process such as salt, soil content, soil-formed materials (clay bricks), water impurities, temperature, air pollution, dust, material quality, porosity etc. [11].

It is not possible to use drinking water at all the time for construction purposes, hence questionable water is preferred in that case. Since the contamination present in the questionable water may become the cause of corrosion and deterioration in concrete structures. So before using those sources of water in the construction, it has to be further tested for its contamination level and then recommended as per the limits for level of impurities prescribed by the IS code.

1.2. Effect of Reinforcement Corrosion

If concrete is carbonated to the depth of the steel reinforcement and a small amount of moisture is present, the steel is likely to corrode. This deterioration is often indicated by fine hairline cracks parallel to the direction of the reinforcement throughout the length of the structural component. Fortunately, as corrosion is fairly uniform, cracking of the concrete cover in normally reinforced or pre-tensioned solid components usually occurs before the steel becomes excessively weak, giving early visual warning of the deterioration [3].

If chlorides are concentrated near the surface of the steel and water and oxygen are abundantly available, severe pitting corrosion may occur. This reduces the cross-sectional area of the bars at these locations, while the remainder of the bar may be left un-corroded. Structural cracks, or honeycombs, can also create conditions favorable to pitting corrosion by allowing the localized ingress of aggressive agents [3].

Reinforcement corrosion and concrete spalling cause a reduction in the ultimate capacity, and more significantly, a reduction in the stiffness and ductility of the R.C section primarily due to the loss of the steel/concrete interfacial bond. The effects of reinforcement corrosion on the behavior of reinforced concrete elements are schematically shown in **Figure 1.**

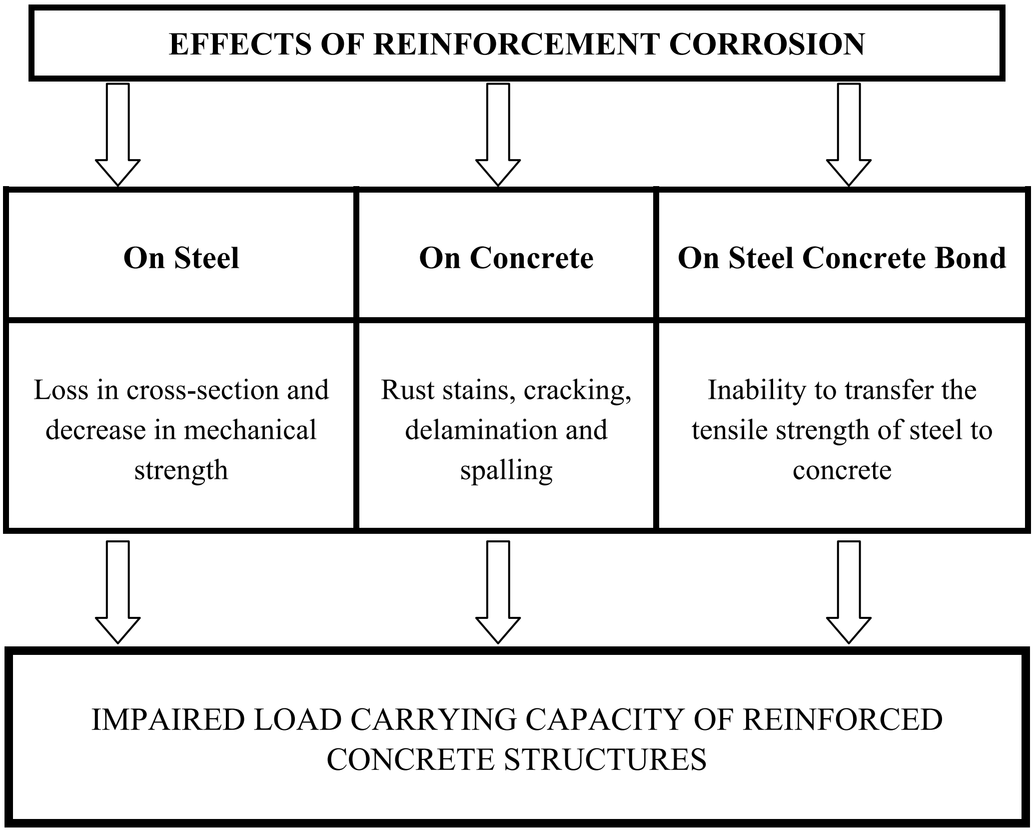

Figure 1. Effects of Reinforcement Corrosion on Reinforced Concrete Structures [3].

2. Corrosion mechanism

There are various type of reactions which assist corrosion process because several elements (sulphate, carbon dioxide, chloride, silicate, oxygen, water) participate in chemical reaction in different ways [12].

Corrosion starts by attacking preventive layer on steel bars, after destruction of that layer concrete becomes highly reactive or ionize for electro-chemical corrosion process [1,13]. Corrosion is not constant at all over reinforcement because at different cross-section and environmental condition (ionic behavior, water, oxygen salts etc.) will be different, which make different intensity of corrosion at different places [5,10,14]. Because of chemical and salts reactions, anions and cations are formed at different portion of reinforced concrete system, which make a full circuit for electrical current supply from one end to another (cathode and anode end). Due to the difference in ions (anode and cathode) salts reacts in the presence of oxygen and creates heterogeneous environment and water solution, which acts as an electrolyte and starts an electrochemical process [7]. **Figure 2** demonstrates the fundamental reaction mechanism of corrosion activity.

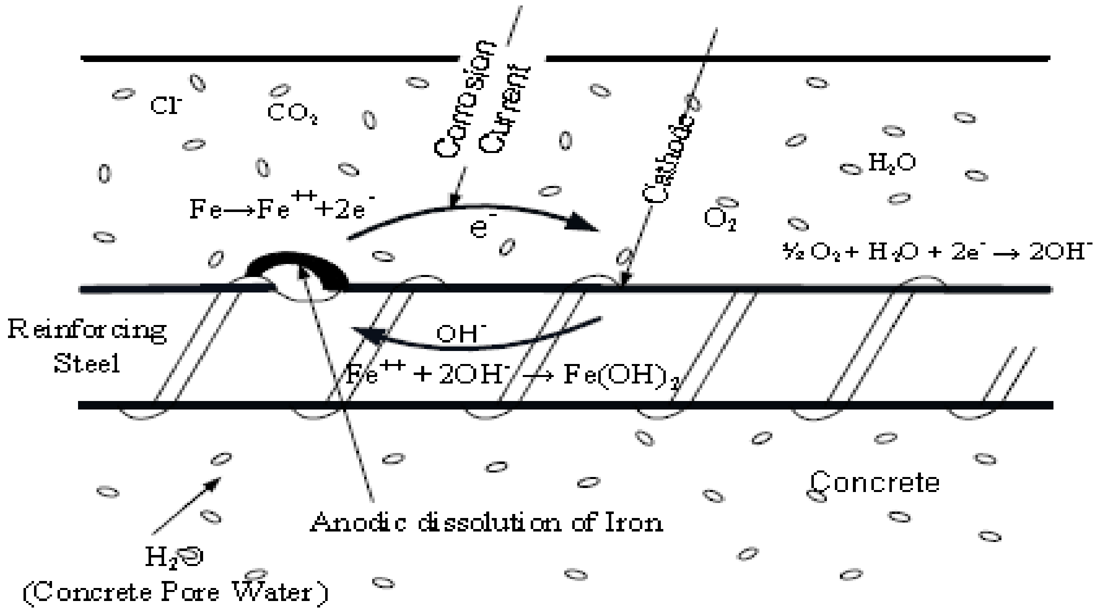

Figure 2. Chloride induced corrosion mechanism surrounding zone of reinforcement [3].

Dispersion of passive film embedded over the reinforcement and occurrence of corrosion reaction occurs due to two causes: chloride salt or carbonation or combination of both processes [7]. Chloride salt also participates in corrosion reaction, usually it reacts with cement and forms calcium chloro-aluminates and calcium chloro-ferrites but due to this reaction chloride-hydroxides ($[Cl^-]/[OH^-]$) ratio reduce which lead to destruction of the passive protective film and imitation of corrosion occurs. After this either chloride makes threshold concentration or pH solution reduces steel with carbonation process [7]. In case of carbonation, carbon-dioxide (CO_2) reacts with water present in pores due to the ingression of moisture from atmosphere or soil, this reaction decreases pH value of concrete to around 9 ranging from 13.5 pH [12].

3. Artificially inducing corrosion in the RC structures

3.1. Impressed current technique

Natural corrosion occurs in years, depending upon different factors like temperature, curing, salts, permeability of concrete etc. Hence, for different corrosion density, different time duration has to be checked but it's not effective to spend that much of time to generate one result. Therefore, the best option for the researchers is to simulate the natural corrosion process with artificial corrosion technique, by means of providing a current source to reinforced sections for a limited duration of time. According to the study carried out by Austin et al. (2004) [15], this technique was very effective and proved to be a quick method to accelerate the corrosion process. However, the reaction and electrochemistry behind the mechanism is somehow different from natural corrosion process but gives an approximately correct and similar report.

Impressed current technique is the most used corrosion enhancement process followed by different researchers. This technique is used to induce a significant degree of corrosion in steel bars adjusting the time limit. This technique is frequently used to study the effect of corrosion on steel bar used in concrete structure to determine the cracking of concrete cover, load-bearing capacity and service life [16]. Different researchers use different type of mix proportion, additional materials (admixture like silica fumes, fly ash etc.) and different cross-section which generates different results. Several Authors like Ahmed (2009) [16], Cairns et al. (2008) [13], Mangat and Elgarf (1999) [9], Ha et al. (2007) [17] used impressed current technique to generate a suitable environment for corrosion process, so that they get proper result for different performance of concrete members, depending upon the flexibility of time period, current density and factors.

Impressed current technique is adopted by an application of constant current from the DC source to the concrete reinforcement, which carries current for a short duration of time. After implementation of current for a given duration, degree of induced corrosion can be measured theoretically using faraday's law and actual loss of steel percentage can be easily calculated. Equivalent current density can be determined through total loss of steel due to corrosion.

Set-up used for inducing current in reinforcement to corrode bars, needs a DC power supply source, a counter electrode and electrolyte solution in which specimen will be submerged. Positive terminal of DC power source is connected to the anode and the negative terminal is connected to the cathode (counter electrode). Constant supply of current is impressed via DC power source from cathode to the reinforcement through the system and electrolyte (normally sodium chloride and calcium chloride solution) helps in easy passage of current within the system [16]. Different authors have used varying type of specimens by adopting different techniques to impress the corrosion current artificially. Some of them are represented below:

Care and Raharinaivo (2007) [18] use different set-up to impress the current in specimen but the phenomena of impressed current technique will be the same. DC power supply is provided in which positive terminal is connected with reinforcement and negative terminal is connected with the counter electrode (cathode plate) by which close loop forms and impressed current flows within the specimen. (See **Figure 3**)

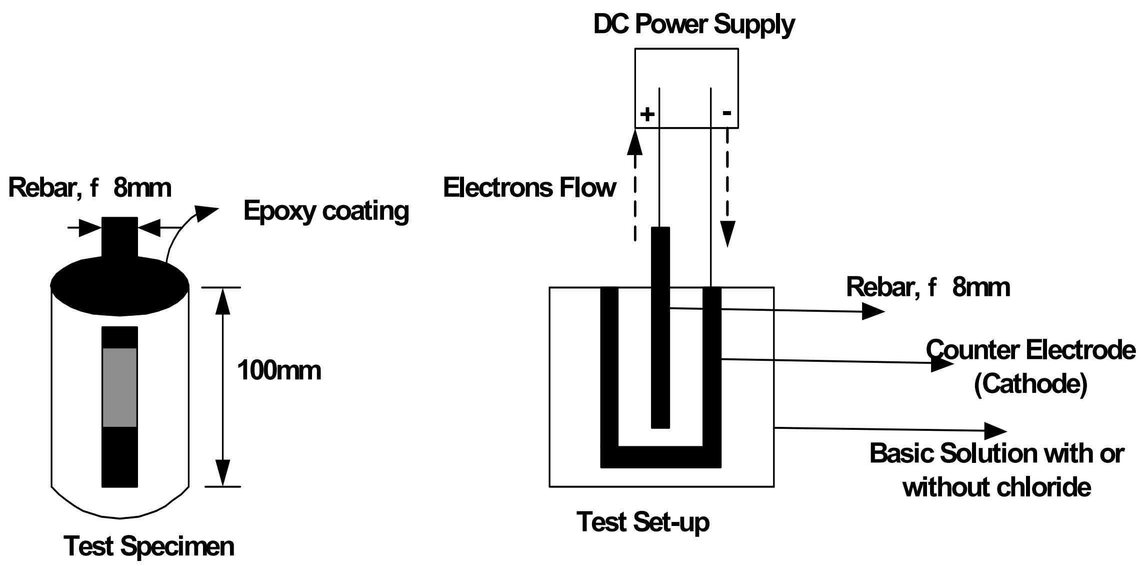

Figure 3. Set-up for accelerating reinforcement corrosion for a typical cylindrical reinforced concrete test specimen [18].

Ahmad et al. (1997) [19] made a series set-up to corrode the entire specimen at the same time so that current density in all specimens is same and on same level of corrosion result can easily be obtained. (See **Figure 4**)

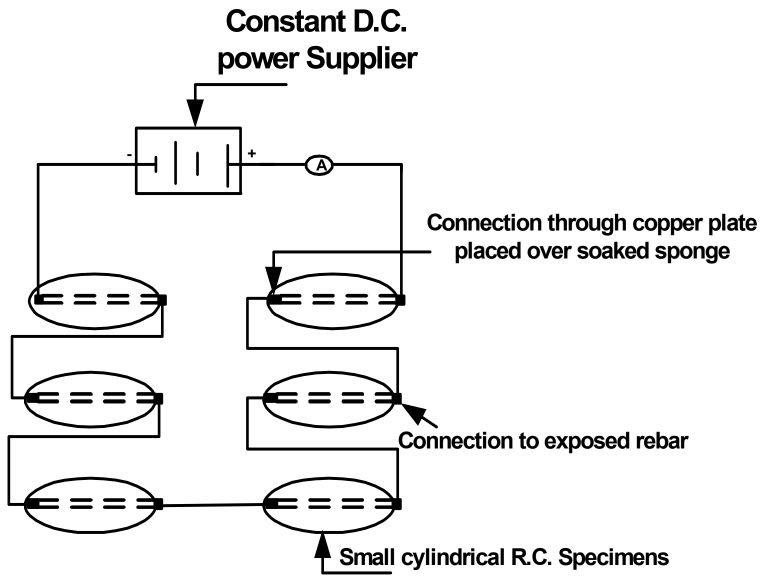

Figure 4. Set-up for accelerating reinforcement corrosion in cylindrical concrete specimens connected in series [19].

Azad et al. (2007) [20] did his experiment with new form of set-up to corrode reinforced concrete structure as shown in **Figure 5.** Similarly, the **s**et-up used by Maaddawy and Soudki (2003) [21] to accelerate corrosion rate in different reinforcements connected in series, as shown in **Figure 6**.

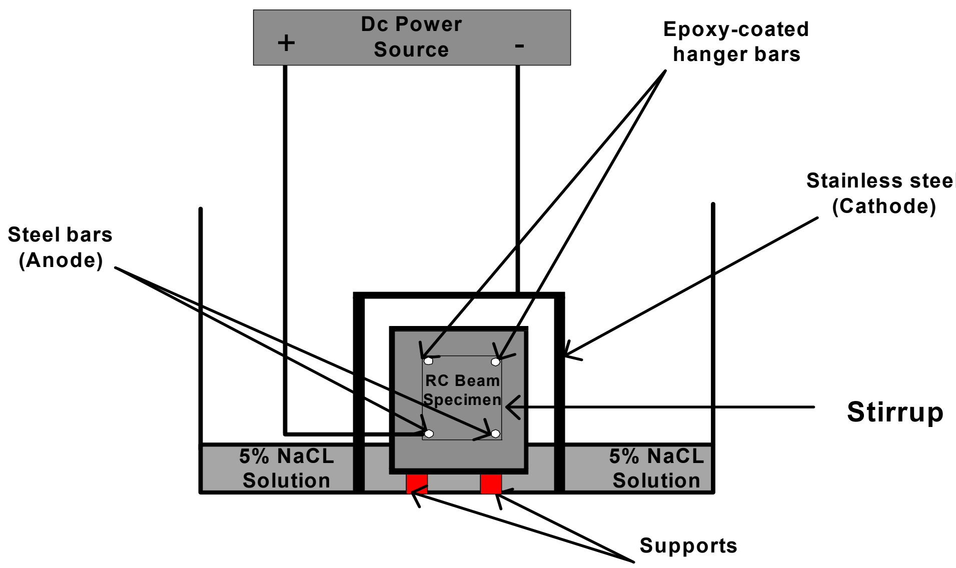

Figure 5. Set-up for accelerating reinforcement corrosion in large-size reinforced concrete beam specimen [20].

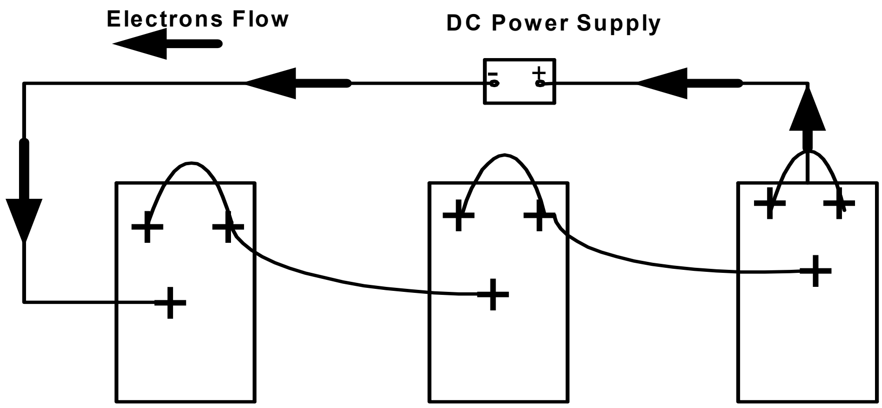

Figure 6. Set-up for accelerating reinforcement corrosion for Reinforced concrete prism specimens connected in series [21].

Certain advantages for impressed current technique were identified by several authors:

(1) In this artificial technique any of the variables which helps in corrosion process can be controlled, for example, change in resistivity of concrete, permeability, current density, time period and (impressed current level) corrosion rate [15].

(2) This process is accurate method for corroding steel bars with effective use of impressed current for de-passivation of bars from years to days and fixing the desired rate of corrosion without compromising the actually formed corrosion product and effect of corrosion [15].

(3) It is also a valid method to study the corrosion process in reinforced concrete and damage effect on concrete cover [18].

(4) Ability to control rate of corrosion which usually varies due to change in the resistivity, temperature and concentration of oxygen in concrete [22].

Although the impressed current technique is precise and accurate to the result, but due to different environmental conditions, it's very tough to simulate the same environment for the corrosion process. Yuan et al. (2007) [23] have found conclusion that the surface characteristic of the corroded reinforcement are found to be different in both impressed current technique and natural corrosion, this difference causes different structural behavior of corrosion on bars. According to Auyeung et al. (2000) [24], theoretical and actual mass losses are not same due to various factors like current density, resistivity, composition, material property, temperature etc.

There are many more researches in practice by different authors in which same phenomena of corrosion is applied but different techniques are being used, for example: Husain et al. (2004) [25] used AC current to accelerate the corrosion process and it took less time than DC current technique. Torres-Acosta et al. (2007) [26] used epoxy resin 3% by weight of 6-mm long carbon fibers added in concrete to accelerate corrosion process. Flaherty et al. (2010) [27] used 1% $CaCl_2$ added in concrete for better corrosion process.

4. Durability and service life prediction of corroded structures

As the time passes, the probability of decrease in durability increases because reinforced concrete structure has different parameter to lose its strength due to the adverse effects of environment, presence of salt, climatic condition etc. [28]. **Figure 7** demonstrates a proper representation of the residual service life (Durability) of any structure. Reinforced concrete mixture is initially free from all type of salts but there are some sources which leads to occurrence of salts in mixture [1,6].

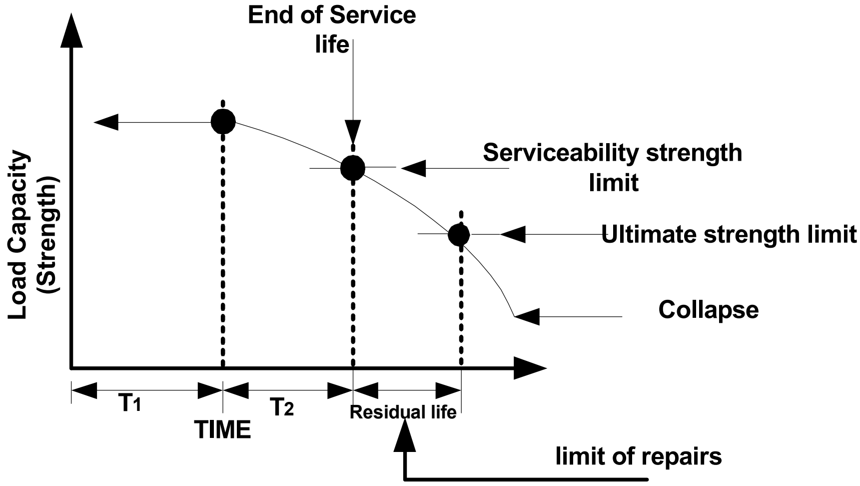

Figure 7. Proposed durability model by Torres-Acosta and Madrid (2003) [28].

If the corrosion is initiated in concrete structures, it reduces service life of the structures and slowly decrease the pH of concrete which helpsc to enhance the corrosion reaction [8,22]. Well mixed and properly cured mortar with less water cement ratio contains less permeability, which decreases rate of percolation of corrosion active elements (chloride, carbon dioxide, moisture, oxygen, sulphate etc.) within the concrete system [6]. After the corrosion process starts, bi-products (ferric oxide, ferrous hydroxide) of corrosion expand and occupy six to ten times more volume than non-corroded steel [1]. Because of the expansion in steel surrounding, cracks are developed which reduce the bonding between coarse and fine aggregate concrete and bond between concrete and reinforcement, which finally leads to the failure of structure.

Corrosion hazard is measured in terms of reduced strength of structural element, loss of cross section of reinforcing bars and bond deterioration, all of which can be associated to corrosion intensity and corrosion duration. Hence, in order to predict the residual strength of any structural element, the main parameters considered for the studies are corrosion intensity and duration of corrosion (i.e. I_{corr} and T). Today, several methods (like Galvanostatic pulse method, Potential monitoring system [34], half cell potential method or LPR method [19], Electrochemical Impedence spectroscopy, Time domain reflectometry (TDR), Ultrasonic guided waves and X-ray diffraction and atomic absorption [7,31])/measurements (equipments like corrosion meter) are available in the field with the help of which corrosion intensity can easily be monitored and the time to initiation and crack propagation can be estimated by using several empirical models predicted for durability designs in the past researches. Practically, several strength prediction model have been proposed in the past decades, which can be utilized either to find the residual flexure capacity of a beam that has suffered corrosion damage or to find the maximum corrosion period for a given level of I_{corr} that can be permitted for a beam at the lowest level of compromised safety. As the empirical method is developed from experimental correlation, it should be recognized that the accuracy of the estimation needs testing in a wider range of $I_{corr}T$ values. For lower corrosion damage, the method is expected to show reasonable accuracy in prediction.

A lot of models have been developed to predict time for different corrosion-induced damages such as cover cracking, loss of steel cross-section area, loss of stiffness etc. Different type of models have different prediction and researches based on different approaches have been presented and discussed [16]. Some of the important parameters are time-variant, nature of corrosion rate, Influence of cover cracking on corrosion rate, Corrosion rate measurement techniques, accounting for variability etc. [32].

According to this model (Refer **Figure 8** for proper understanding), age of structure is inversely proportional to corrosion. Due to corrosion active elements likes salts and carbonation process, pH value decreased due to which adverse environment is created within the concrete system which reduces the strength of reinforcement and concrete by enhancing corrosion reaction [31].

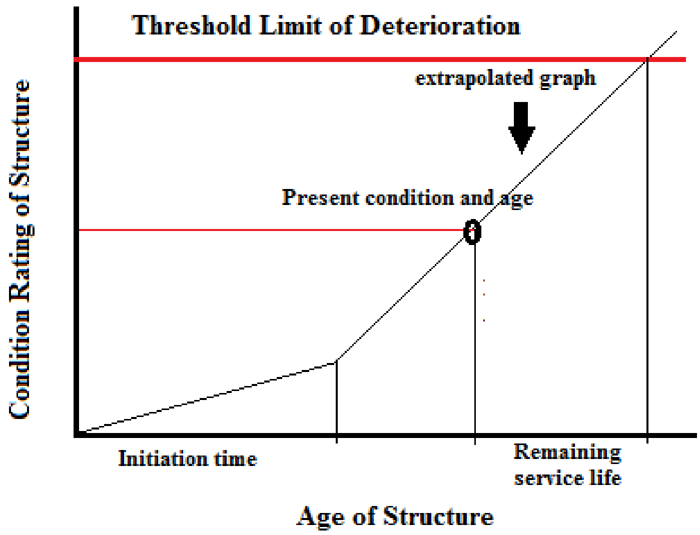

Figure 8. Prediction of service life [31].

General ideas obtained from several researches related to corrosion phenomena are as follows:

(1) The loss of bond between corroded bars and concrete [11].

(2) Corrosion current density increases with increase in the moisture content and reaches a peak value, after which it decreases [22].

(3) The degree of corrosion, $I_{corr}T$, and rebar diameter D, have significant effects on residual flexural strength, R with increase in $I_{corr}T$ and D, there is a decrease in R [6].

(4) The level of chloride ions required to initiate corrosion in concrete corresponds to 0.10% soluble chloride ion by weight of cement [5].

(5) For the artificial corrosion normally 3.5%–5% NaCl or 2%–10% calcium chloride is preferred for the best ionic water compound, power supply with a current intensity of 1–4 mA/cm^2 was used to induce the corrosion for the best result according to the experiment [9,11].

5. Residual flexural strength of corroded beams

Several researchers have focused their research towards estimating the residual flexure strength of corroded beams by adopting varying parameters with the view to ascertain its effect on the strength. Few of the reports obtained by some of the authors are reported here in this study. The details of the experimental parameters adopted by several authors for residual flexure strength are summarized in **Table 1**.

Table 1. Summary of the experimental parameters adopted by several authors for Residual Flexure Strength.

S. No	*Author*	*Dimension (mm)*	*Salt percentage*	*Corrosion Parameters*		*Water cement ratio (%)*	*Additional Information*
				Current density	*Corrosion duration or %*		
1	Mangat and Elgarf (1999) [9]	100×150×910	NaCl 3.5%	1, 2, 3 and 4 mA/cm^2	12–384 hour	0.53%	Impressed current technique used for artificial corrosion
2	Cairns et al. (2008) [13]	150×200×1875 150×200×1375	NaCl 3%	0.06 mA/cm^2	120 day	Not reported	Impressed current technique used for artificial corrosion
3	O'Flaherty et al. (2008) [27]	100×150×910	$CaCl_2$ 3.5%	1 mA/cm^2	0–15%	Not reported	Galvanostatic corrosion process used for artificial corrosion
4	Gu et al. (2010) [33]	150×200×2200	NaCl 5%	1 mA/cm^2 and 0.2 mA/cm^2	Not reported	0.55%	galvanostatic corrosion process used for artificial corrosion
5	Shannag and Al-Ateek (2006) [34]	100×150×1000	NaCl 5%	3 mA/cm^2	Not reported	0.67%	Different type of fiber is used and galvanostatic corrosion process used for artificial corrosion
6	Mohammed Ali (2014) [35]	150×150×750	NaCl 8%	Not reported	30 days	0.45%	galvanostatic corrosion process used for artificial corrosion, the bond reduction about 80%
7	Torres-Acosta et al. (2007) [36]	100×150×1500	NaCl 3%	0.08 mA/cm^2	40, 80 and 200 days	0.5%	3% of 6-mm long carbon fibers is used, galvanostatic corrosion process used for artificial corrosion
8	Azad et al. (2010) [37]	200×215×1100 200×265×1100 200×315×1100	NaCl 5%	1.78 mA/cm^2	Different duration of corrosion adopted from 2 days to 20 days	0.45%	partially immersed at depth of 120mm, Impressed current technique used for artificial corrosion
9	Tachibana et al. (1990) [38]	150×200×2000	NaCl 3.3%	0.5 mA/cm^2	3, 6, 10, 15 days	Not reported	Only tensile reinforcement used, Artificial corrosion by Impressed current technique
10	Andrade et al. (1993) [39]	200×200×2000	NaCl 3.5%	10, 100 $\mu A/cm^2$	4, 6, 14 and 35 days	Not reported	galvanostatic corrosion process used for artificial corrosion

5.1. Report by O'Flaherty et al. (2008) [27]

Authors designed reinforced concrete beams to provide ductility in under-reinforced concrete section i.e. moment of resistance M_c (compressive) is greater than moment of resistance $M_{t(0)}$ (tensile). In their study, they discussed about the influence of $M_{t(0)}/M_c$ on the residual flexural strength of corroded steel in beams and determined detailing parameters on $M_{t(0)}/M_c$ ratio. A total of 12 beam having cross-section 100 mm × 150 mm × 910 mm with two bar of different diameter 8 mm, 10 mm and 12 mm were being used in the test. All specimens were designed for flexural failure by providing sufficient links to prevent shear failure. Beams were designed to target corrosion rate having increment of 5% from 0-15%. Anhydrous calcium chloride ($CaCl_2$) was added to the mix to enhance corrosion process which initiates the reaction process. Controlled corrosion process was done at corrosion rate of 0%, 5%, 10%, 15% after setting of concrete at 28 days, the sample was tested after 28, 42, 56 and 63 days respectively. The study concluded that:

(1) Higher degree of under-reinforcement lower $M_{t(0)}/M_c$ of reinforced concrete beams results in lower loss of strength caused by reinforcement corrosion.
(2) Cover provided to the reinforcement does not have an effect on $M_{t(0)}/M_c$.
(3) An estimate of the residual tensile moment of resistance of corroded beams within the limits of the test data given in the paper can be obtained from the expression (**Eq 1**)

$$M_{t(corr)} = \frac{M_c \times \alpha\ (corr\%)}{100} \times M_{t(0)} \quad \textbf{(1)}$$

where, $M_{t(corr)}$ = Moment of resistance of the corroded beam in the tensile zone,
M_c = Maximum moment of resistance of concrete in the compression zone,
α = slope of $M_{t(corr)}/M_c$ against percent of corrosion,
$M_{t(0)}$ = Moment of resistance of the control beam in the tensile zone.

5.2. Report by Gu et al. (2010) [33]

Gu et al. (2010) [33] constructed twelve beams where three beams were corroded by natural corrosion process and others by artificial corrosion process. All beams had a cross-section of 150 mm × 200 mm × 2200 mm having two deformed rebar, each in a diameter of 12 mm, 14 mm and 16 mm and concrete cover was 25 mm. Beams were casted and cured in natural condition for 28 days, after which they were immersed in 5% sodium chloride solution. Two different current densities were adopted as 1000 μA/cm^2 and 200 μA/cm^2 for different beam groups. They concluded with certain points:

(1) High impressed current density can accelerate the deterioration process of the reinforced concrete beams before cover cracking.
(2) Load carrying capacity and stiffness of beams decreases with the increase in the corrosion degree.
(3) A practical calculation method for RLCC is developed and is expressed in **Eq 2**,

$$M_u = \beta_1 f_c ba\left[d - \left({}^{a}/_{2}\right)\right] \quad \textbf{(2)}$$

where, $a = (f_{yc}A_{sc})/(\beta_1 f_c' b)$, M_u = ultimate bending strength of corroded beam, $\beta_1 f_c'$ = equivalent stress intensity, b = width of beam, a = depth of the equivalent stress block, d = effective depth, f_{yc} = yield strength, A_{sc} = residual cross-sectional area of the corroded tensile bar.

5.3. Report by Shannag and Al-Ateek (2006) [34]

Authors constructed a total of 30 concrete beams in ten different groups having dimensions of 100 mm × 150 mm × 1000 mm, after corrosion cover were replaced with fiber reinforced composite material for the high performance. Beams were immersed to a depth of about 100 mm in 5% NaCl solution. Stainless steel plate of 1.4 mm × 60 mm × 350 mm was immersed in the solution to be used as a counter electrode. A power supply with a current intensity of 3 mA/cm^2 was used to induce the corrosion. Four point bend test was performed for their load-deflection relations. They concluded that:

(1) The beams casted with fiber reinforced concrete cover zones showed a substantial improvement in flexural performance in comparison with the properties of the unreinforced matrix.
(2) Beams cast with a 50/50 blend of brass-coated and hooked steel fibers BHFRC achieved the highest flexural load capacity under different corrosion rates, ranging from 0% to 5.5%.
(3) Reinforcement corrosion in beam specimens had a marked reduction on stiffness and deflection of beams, flexural load capacity, and ductility.
(4) Beams casted with GFRC cover zone showed the best performance compared to other beams.

5.4. Report by Mohammed Ali (2014) [35]

In this study, artificial as well as natural corrosion processes were carried out to corrode steel bar by immersing it in 8% of NaCl salt water for 30 days inside the lab and was placed outside to corrode naturally for 40 days. Two types of rebar were used having diameter 10 mm and 20 mm. For the artificial treatment corrosion ratio was measured with the sample before and after corrosion by removing corrosion layer (rust) and by comparing new and old diameter of rebar. In the second stage sixteen reinforced beam of dimension 150 mm × 150 mm × 750 mm without removing corroded layer were casted. Concrete mix used to cast the beams consisted of cement, coarse aggregate, fine aggregate, and water with a corresponding proportion of 1:2:4:0.45 by weight of cement. The beams were tested under increasing two point-load at mid one-third span till failure.

In this study they concluded that:

(1) The percentage of increasing load for corrosion bar was found to be 144.3%, 141.7%, 124.3% for 30 days, 90 days and 120 days.
(2) Earlier non-corrosion rebar beams gave maximum load, moment, displacement and curvature, but this situation changed later.
(3) The constant corrosion layer was found by artificial corrosion and without any local pits as discovered by other researchers [40–42]
(4) Beams of high corrosion rebar gave better results as compared to beams of low corrosion rebar and high corrosion rebar 10 mm gave better results than same corrosion rebar 12. This could be produced by improving the bond characteristics at the steel-concrete interface, where the deterioration of protective layers took place.

5.5. Report by Torres-Acosta et al. (2007) [36]

The authors constructed twelve beams having cross section of 100 mm × 150 mm × 1500 mm, concrete beams reinforced longitudinally with one #3 (10 mm in diameter) rebar were made. Chloride ion contamination of 3% by weight of cement (~12 kg/m^3) was obtained by adding NaCl (table salt) during concrete mixing. The conductive polymer was made using a commercially available two-component epoxy resin and adding 3% by weight of 6-mm long carbon fibers as tested elsewhere. The anodic current density of 80 μA/cm^2 (electrical current divided by rebar's geometric surface area) was applied for a period of ~40, ~80, or ~200 days according to the desired nominal corrosion damage 5, 10, and 15% rebar radius loss. Wet sponges were used on top of the beams to maintain humidity throughout the beam length. In this study they concluded that:

(1) The corrosion-induced concrete crack propagation was enhanced if dry rather than wet environment is used during the accelerated corrosion stage.
(2) Wet environment during corrosion acceleration enhanced pit formation at the rebar surface;
(3) A decrease of as much as 60% in the flexure load capacity values was observed with only 10% of x_{AVER}/r_0, where r_0 = rebar radius.
(4) PIT_{MAX}, not the x_{AVER}/r_0 ratio, was the most important parameter affecting flexural load capacity reduction in corroded beams.
(5) The flexure load capacity diminished mainly due to the formation of pits on the rebar surface, which in this investigation were as deep as 73% of the original rebar diameter.

5.6. Report by Azad et al. (2010) [37]

They constructed forty eight beams in which twelve beams were analyzed with two-step analytical process to determine the residual flexural strength of corroded beams and remaining beams were subjected to a varying degree of corrosion damage using accelerated corrosion and then they were tested in a four-point bending test to determine their residual flexure capacity. The beam sizes used in the experimental work width was constant 200 mm and depth were varying in three phase 215 mm, 265 mm and 315 mm depth with length of beam 1100 mm. They were subjected to accelerated corrosion through impressed current technique by providing 1.78 mA/cm^2. Three different beam depths and tension bars of two different diameter of 16 mm and 18 mm were used in the experiment. The concrete specimens were partially immersed up to a depth of about 120 mm in 5% sodium chloride solution in a tank. In this study they concluded that:

(1) The percentage-wise loss of metal was smaller for a large diameter bar in comparison with that for smaller diameter bar at a constant $I_{corr}T$.
(2) The study reaffirmed that, at lower value of $I_{corr}T$, the residual flexural strength of a corroded beam can be predicted with a reasonable accuracy by considering only the reduced cross-sectional area of tension reinforcement A_s'.

5.7. Report by Mangat and Elgarf (1999) [9]

They constructed total of 111 beams within nine groups of different parameters. Two external power supplies were given to enhance corrosion process and to observe the damage occurred before and after corrosion process. The cross-section of beam was 100 mm × 150 mm × 910 mm and all group consist of two deformed bar of 10 mm except group 3 and 7 in which 8 mm bar was used, also group 1 and 4 have stirrups of 6 mm diameter and 70 mm spacing, no stirrups was provided in other groups. The corrosion process took place in plastic tank having 3.5% of NaCl solution used as electrolyte. Group 1, 2 and 3 were controlled beam and other group was artificially corroded with a current intensity of 1, 2, 3 and 4 mA/cm^2 for different duration of corrosion ranging between 12 to 384 hours.

As corrosion starts to occur, strength decreased because chemical reaction takes place on the surface of steel and forms iron oxide which decreases the density of steel and increases its surface area due to which bonding between concrete and steel gets disrupted leading to a loss in strength. In this study they concluded that:

(1) Period of pre-curing up to 1 year before corrosion had no effect on flexure strength.
(2) Due to corrosion effect on cross-sectional area, insignificant effect was observed on residual flexure strength of beam.
(3) Decrease in strength was due to loss in cross-sectional area of steel or breakdown of interfacial bond of concrete and steel.
(4) Residual flexure strength after corrosion can be obtained by using **Eq 3**,

$$B\ percent = \left[1 - sin^2\left(2.312\frac{T}{D}ilni\right)\right] \times 100 \quad \textbf{(3)}$$

where, B percent = percent of flexure strength of control beam
T = time elapsed in year after corrosion initiation
D = reinforcing bar diameter (mm)
i = rate of corrosion (μA/cm^2)

5.8. Report by Cairns et al. (2008) [13]

Cairns et al. (2008) [13] developed four type of beam with different cross sections, concrete cover was kept as 20 mm and addition of 4% sodium chloride within the concrete mix was adopted. Shear span/effective depth ratio for shorter span was taken 2.7 in both cross-section beams. Different diameter of bar 6, 8, 10, 16 mm was used for stirrup and main bar for different beams. Beams were corroded in a shallow tank having 3% of salt solution and constant impressed current density of 0.06 mA/cm^2 was applied. Different beam failure was observed due to bond failure or flexure failure. In this study they concluded that:

(1) Beams with high steel ductility plan round bar gave approximately 10% loss in cross-section, also 0.3 mm corrosion penetration and having crack of 1 mm.
(2) Bond tests with relatively short bonded lengths do not allow bars to attain yield, and hence limited relevance to the bond failures was observed in the beam tests.
(3) The area of links used in these tests was higher than the minimum area specified for shear in BS 8110. This may have been beneficial to residual strength.

5.9. Report by Andrade et al. (1993) [39]

They constructed four specimen with different factors, by placing bar at corner with 20 mm and 30 mm cover in specimen 1, reinforced bar was provided at center top portion having 20 mm cover of specimen 2, 3 and 4, bar is placed at center top with 30 mm cover. Specimen with cross-section of 150 mm × 150 mm × 380 mm and 16 mm diameter bar was used in the experiment. 3% of $CaCl_2$ was mixed into the concrete for better corrosion and cured specimen for 28 days and 0.5 water cement ratio used in this study. Current density of 100 μA/cm^2 was used for specimen Type 1, 2, 3 and for specimen Type 4, current density of 10 μA/cm^2 was adopted for artificial corrosion. Time duration for corrosion was taken as 4, 6, 14, 35 days after 28 days of curing. In this study they concluded that:

(1) Volume of steel increased when corrosion process took place and rust formed.
(2) Crack width rate increase due to corrosion is proportional to relative increase in steel volume.
(3) Crack width expanded slowly
(4) Crack propagated from where bond of concrete particles were weakened because of porosity, corrosion, internal volume change of steel, adverse climate etc.

In spite of such a huge findings particularly in case of residual flexure strength, researches illustrated the prediction work using an empirical method to be under confident and does not provide much promising results which oriented the researchers to check its applicability contradicting towards producing a more generalized solution. Following which, studies have been carried out towards predicting the residual strength parameters using soft computing techniques (ANN, GA, Fuzzy Logic, SVM, etc.) which depicted a substantial enhancement in the predicted outcome and has proved to be a more reliable approach.

With the limitations in the experimental and theoretical methods, the pursuit for cost-effective, easy to use and adaptive models that offer efficient generalization capability to new cases continues. With the huge amount of data generated from various experiments over the years, robust data mining techniques that were based on computational intelligence (CI) and machine learning paradigms are hypothesized to be capable of overcoming the limitations of the conventional methods [43]. Artificial neural networks (ANN) are well established technologies being adopted in a variety of application ranging from pattern recognition to optimization. One of the attractive features of ANN is their ability to perform non linear, multi dimensional interpolations. This feature of ANNs makes it possible to capture the existing non linear relationships between input and output parameters [44,45]. ANN is the most commonly used of the CI techniques in various engineering application areas [46,47].

6. Residual shear strength of corroded reinforced concrete beams

Several researchers have focused their research towards estimating the residual shear strength of corroded beams by adopting varying parameters with the view to ascertain its effect on the strength. Few of the reports obtained by some of the authors are reported here in this study. The details of the experimental parameters adopted by several authors for residual flexure strength are summarized in **Table 2**.

Table 2. Summary of the experimental parameters adopted by several authors for residual shear strength.

Experimental Details for Residual Shear Strength							
				Corrosion Parameters		*Water*	
S. No.	*Author*	*Dimension (mm)*	*Salt percentage*	*Current density*	*Corrosion Duration (days)*	*cement ratio (%)*	*Additional Information*
1	Rodriguez et al. (1997) [11]	150×200×2300 150×200×2050	CaCl2 3.5%	100 µA/cm^2	100 to 200 days	0.5	galvanostatic corrosion process used for artificial corrosion
2	Flaherty et al. (2010) [27]	100×150×910	CaCl2 3.5%	1 mA/cm^2	42, 48 and 45 days	Not reported	1% $CaCl_2$ added in concrete for better corrosion process.
3	Imam and Azad (2016) [48]	140×220×1150 150×240×1100	NaCl 3%	2 mA/cm^2	6 and 10 days	0.4	Designed to fail in shear only; a/d ratio is taken as for group A = 1.76 and for B = 1.57
4	Xue and Seki (2010) [49]	120×240×220	NaCl 3%	1 mA/cm^2	Not reported	0.48	Immersed in 10% ammonium hydrogen citrate solution; a/d ratio is taken as 1.5–4.0
5	Juarez et al. (2011) [50]	200×350×2000	NaCl 3.5%	0.1 mA/cm^2	80 and 120 day	Not reported	a/d ratio adopted as 2
6	Xia et al. (2011) [51]	120×230×1200	NaCl 5%	0.2 mA/cm^2	Cycle of drying-wetting process is in 3 to 4 days	0.53	At different duration different bar is Galvanized
7	Suffern et al. (2010) [52]	125×350×1800 (Clear span 1500)	Chlorides 2.3%	0.45 and 0.15 mA/cm^2	21, 60 and 120 days	0.55	a/d ratio adopted as 1, 1.5 and 2

6.1. Report by Imam and Azad (2016) [48]

Imam and Azad (2016) [48] constructed seventeen beams having two different cross-sections of 140 mm × 220 mm × 1150 mm and 150 mm × 240 mm × 1150 mm with two different corrosion duration 6 days and 10 days, where all beams were designed to fail in shear. Out of seventeen beams, thirteen were kept as corroded and four as un-corroded (controlled). The bottom cover of concrete was 50 mm (cover over stirrup is 32 mm) and the cover side to the stirrup was 40 mm, spacing of stirrup was kept below d/2 spacing. Shear span to effective depth ratio for group A, a/d = 1.76 and Group B specimens' a/d = 1.57. If a/d ratio is greater than 1.5 but less than 2.0, can be classified as shallow beam. The mix design used for all specimens consisted of cement content of 370 kg/m^3 (ASTM Type I Portland cement), coarse to fine aggregate ratio of 1.46 (by mass) and water cement ratio of 0.4 (by mass). The concrete specimens were immersed up to a depth of about 160 mm in 3% sodium chloride solution in a tank and using corrosion current density of 2.0 mA/cm^2. The corrosion period was chosen as 6 and 10 days to induce low to medium degree of corrosion damage. All beams, corroded and un-corroded, were tested as simply supported beams of 900 mm span using four-point loading under a universal testing machine. In this study they concluded that,

(1) The key parameter for the corrosion damage is the corrosion activity index, $I_{corr}T$. Metal loss, amount of crack-induced damage and the loss of shear strength increase with increasing $I_{corr}T$.

(2) Crack damage. Because of corrosion-induced cracking, the concrete cover does not fully contribute to shear strength, unlike the core within the confinement of steel which essentially remains un-cracked and undamaged.

6.2. Report by Xue and Seki (2010) [49]

They casted several beams with cross-section of 120 mm in width, 240 mm in overall height and 220 mm in effective depth. Ultra-high strength steel bars (f_y = 706 N/mm^2) were used as longitudinal bars to acquire strong (safe) flexural strength. Apart from the specimens with 2.6 as a/d ratio, which was thought to be sensitive to failure mode, specimens with four other different a/d (1.5~4.0) were also fabricated. The specimens were immersed in 3% NaCl solution with a current density of 1 mA/cm^2. They proposed an empirical relations for shear capacity which is given below as **Eq 4**:

$$V_u = \alpha_1 \times V_u \quad \textbf{(4)}$$

Where, $\alpha_1 = f(a/d\,, C)$

$V_{u\text{-}eval}$ = shear capacity of corroded RC beams,

V_u = shear capacity of sound RC beams,

C = average $_{mass}$ loss of corroded longitudinal bars,

a/d = shear-span-to-effective-depth-ratio.

In this study they concluded that:

(1) The investigational results indicate that the shear behavior of RC beams was influenced not only by the corrosion level of the longitudinal bars but also by the shear-span-to-effective-depth-ratio (a/d).
(2) A modified equation capable of calculating the shear capacity of RC beams with corroded longitudinal bars was proposed and its validity was proved.
(3) The shear behavior of RC beams with corroded longitudinal bars was influenced not only by the corrosion level of longitudinal bars, but also by the shear-span-to-effective-depth-ratio.
(4) When the shear span to effective depth ratio was above 3.0, the prediction of the shear capacity of RC beams with corroded longitudinal bars using current shear equation will result in an unsafe over valuation.

6.3. Report by Juarez et al. (2011) [50]

They constructed two series of eight reinforced concrete beams measuring 200 mm × 350 mm × 2000 mm. Ready-mix concrete with f_c = 21 MPa was used, with a slump of 85 mm. Five #16 longitudinal bars (15.9 mm in diameter) were covered by a resin-based epoxy anticorrosive paint, #8 stirrups were used (7.6 mm in diameter). Continuous wet/dry cycles were applied by wetting the zone for 3 days with a sponge soaked in a 3.5% NaCl solution followed by 4 days air drying. A 100 $\mu A/cm^2$ galvanic current was applied for 20% and 50% loss of shear strength for either 80 (moderate) or 120 (severe) days in order to reach different levels of corrosion in the stirrups. In this study they concluded that:

(1) Ultimate shear strength was mainly affected by moderate and severe level of worse corrosion attack.

(2) Normal beam showed 30% more ultimate shear strength than the corroded beam.
(3) Additionally, beam ductility was affected by levels of moderate and severe deterioration of stirrups, and this was evident due to the brittleness and sudden failure observed during beam testing.
(4) The average remaining section based on critical diameter of the stirrups has good performance of ultimate shear strength in future.
(5) Incipient deterioration level did not significantly affect concrete/steel adherence because there was no reduction in stirrup diameter compared to the control beams.
(6) The cross section calculated with the critical diameters of the stirrups, for both the moderate and severe deterioration levels, represented a robust index to predict the ultimate shear strength of reinforced concrete beams.

6.4. Report by Xia et al. (2011) [51]

In their experiment, a total of 18 reinforced concrete beams in three different groups were tested which included 15 corroded beams and three un-corroded beams having a cross-section of beam 120 mm × 230 mm × 1200 mm and 11 stirrups (spacing 100 mm) of 6 mm diameter steel bars. Average compressive strength of concrete was found as 42.5 MPa with water/cement ratio was 0.53 and having different diameter of bars. The corrosion procedure can be divided into two phases, namely, the electro-migration phase and the wetting–drying cycle phase, 5% NaCl solution was put in the sponge to put the concrete moist. A constant voltage of 30 V was applied between the outside stainless steel nets and the embedded stainless steel sheets using a DC power source. For the purpose of an accelerated corrosion, a current density of 2 A/m^2 was applied during the wetting process through the stirrups. In this study they concluded that,

(1) The severe corrosion level of the reinforcing steel bars, wider the average crack width and so will be the maximum crack width of the concrete cover.
(2) The maximum crack width increased compared with the average crack width. The maximum crack width increases more quickly with the average crack width.
(3) The decrease of the stiffness was insignificant when the applied load was relatively low. It was only when the applied load exceeded 20%–30% of its ultimate load, the stiffness loss caused due to the reinforcing steel corrosion became significant.
(4) As the corrosion level became severe, shear failure mode of the beams may change from concrete crushing to stirrup failure. This was attributed to the cross-section loss of stirrup bars, which became severe as the corrosion level increased.

6.5. Report by Suffern et al. (2010) [52]

They constructed 15 beams having cross-section 125 mm × 350 mm × 1850 mm with a clear span of 1500 mm and cover of 22.5 mm. Shear span-to-depth ratios (a/d ratio) was taken as 1.0, 1.5, 2 and different corrosion duration were adopted as 21, 60 and 120 days with varying corrosion intensity as low, medium and high. The amount of salt added as 2.3% chlorides by mass of cement. The DC power supply can apply a maximum current of 500 mA with an accuracy of 1%. In this study they concluded that,

(1) The shear strength reduction was up to 53%. Furthermore, the reduction in shear strength due to the corrosion was found to be greater at smaller shear span-to-depth ratios.
(2) Corrosion of the stirrups produced relatively uniform mass loss along both legs of an individual stirrup.
(3) The measured average crack width due to corrosion was 0.3, 0.4 and 0.8 mm in the low, medium, and high corrosion level specimens,
(4) The corroded specimens with the lowest shear span to depth ratio experienced the highest reduction in ultimate shear strength.

6.6. Report by Flaherty et al. (2010) [27]

A sixth study was conducted by Flaherty et al. (2010) [32] in U.K. Beams were constructed having a cross-section of 100 mm × 150 mm × 910 mm. There were two set of beams, 1A beams were reinforced with 2T8 main steel and 6 mm mild steel shear reinforcement and type 1B beams has main reinforcement consisted of 2T12. Target corrosion of 0%–15% of cross sectional area was applied to the shear reinforcement of these beams in 5% increments. Cover was 50 mm to the shear reinforcement for the beams and 65 mm of spacing was provided in stirrups. The control specimens (0% corrosion) were tested at the age of 28 days. The corroded beams were tested at 42, 48 and 45 days of age, so as to reach the target corrosion of 5, 10 and 15%. The corrosion process took place in a plastic tank where a 3.5% $CaCl_2$ solution was used as the electrolyte. A constant current density of 1 mA/cm^2 was passed through the reinforcement. They concluded that:
(1) This was more pronounced for the beams with a higher $M_{t(0)}/M_c$ ratio (lower degree of under-reinforcement) and this should be taken into account at the design stage.
(2) The predominant failure mode for under reinforced concrete beams exhibiting low degrees of shear reinforcement corrosion is flexural (<18.7% in this investigation).
(3) Shear failure occurred only at higher degrees of shear reinforcement corrosion (>18.7% in this investigation).

6.7. Report by Rodriguez et al. (1997) [11]

They constructed two groups, each having six beam of dimensions 150 mm × 200 mm × 2300 mm and 150 mm × 200 mm × 2050 mm, where bending tests were performed for shear as well as flexure. Only bottom reinforcement were corroded in type one beam and in second type, all reinforcements were corroded. Different type of bars were used for different beam segment having diameter 6, 8, 10 and 12 mm for different corrosion activities. The current density of 100 $\mu A/cm^2$ impressed into the bars for 100 to 200 days of time period. Cracks were developed in both flexure and shear portion of about 0.2 mm to 0.6 mm on average. In this study they concluded that:
(1) Corrosion process degraded the reinforcement directly and also degrade the performance of concrete by creating cracks, reducing strength at ultimate strength.
(2) Pitting at links creates most relevant damage in reinforced concrete section.
(3) Because of cracking and spilling created by corrosion, deterioration of concrete cover occurred.
(4) Reduction in steel bar diameter due to corrosion can be measured by corrosion intensity (I_{corr}) in reinforced concrete structure.

7. Findings obtained by several authors

Different type of materials, water cement ratios, current density, corrosion rate, exposure condition and time period etc. are several varying parameters adopted by past researchers to establish an understanding about the weight or cross-sectional loss, cracks, reduction in strength and degradation in concrete quality due to corrosion activity inside the reinforced concrete structures. The details of those findings have been summarized in **Table 3**.

Table 3. Findings and observations obtained by different Authors.

No	*Author*	*Accelerated Corrosion Intensity*	*Exposure Time*	*Sectional loss (%)*	*Crack Width (mm)*	*Strength loss (%)*
1	Andrade et al. (1993) [39]	10, 100 µA/cm^2	4, 6, 14 and 35 days	up to 0.25 cross section loss %	0.05–0.3 mm	20%–50% flexure strength loss
2	Cabrera (1992) [53]	-	40 day	0.8–9.2 % Cross section loss	0.06–0.46 mm	Not reported
3	Imam and Azad (2016) [48]	2 mA/cm^2	6 and 10 days	~ 76% cross section loss in stirrups	Not reported	57%–62% Shear strength loss
4	Torres-Acosta et al. (2007) [26]	0.08 mA/cm^2	40, 80 and 200 days	20–75% mass loss in main bars	0.5–3 mm	30%–75% Flexure strength loss
5	Mangat and Elgarf (1999) [9]	1–4 mA/cm^2	12–384 hour for diff. groups and beams	2.5–10% 1.25–5% Mass loss	0.1–0.4 mm	~25 % flexure loss
6	Rodriguez et al. (1997) [11]	100 µA/cm^2	100–200 days	11.6–22.1% Cross sectional loss in stirrups	0.2–0.6 mm	Flexure loss observed in the range of 40–50%
7	Haung and Yang (1997) [54]	5 A/mm^2	126 hour	<1 % Mass loss	Not reported	Not reported
8	Tachibana et al. (1990) [38]	0.5 mA/cm^2	3–15 days	2.5–12% Mass loss	0.1–0.75 mm	Not reported
9	Suffern et al. (2010) [52]	0.45 and 0.15 mA/cm^2	21, 60 and 120 days	2.5–18.7% mass loss	0.3–1 mm	53%, 50% and 23% depending on a/d ratio as 1, 1.5 and 2
10	Gu et al. (2010) [33]	1 mA/ cm^2 and 0.2 mA/cm^2	corrosion duration from 225–3920 hr. for different groups	3.4–32.5% Mass loss	0.59–5, spalling is also observed in some beams	40%–55% flexure strength loss
11	Uomoto and Misra (1998)][55]	280–380 µA/cm^2	7 to 14 days	1–2.4% Mass loss	Not reported	4–17% Strength loss
12	Xue and Seki (2010) [49]	2 mA/cm^2	Not reported	0%–18% Mass loss	>1 mm	<70% Strengh loss
13	Juarez et al. (2011) [50]	0.1 mA/cm^2	80 and 120 day	23%–42% cross sectional loss in stirrups	~15 mm	20% to 50% loss of shear strength
14	Xia et al. (2011) [51]	0.2 mA/cm^2	Cycle of drying-wetting process is in 3 to 4 days	0–54.15% cross section loss	0–0.19 mm	Not reported
15	Cairns et al. (2008) [13]	0.06 mA/cm^2	120 day	4.4–11.5% Mass loss	0.20–1.00 mm	5–14% Loss in yield strength

8. Conclusion

Corrosion of reinforced concrete is a major issue for construction work all over the world. Thousands of paper are being published every year in different journals only to know every possible behavior, factors, monitoring technique, different simulation and other phenomena so that possibility of more accurate results can be obtained which shows full behavior of structure. According to all studies which were done by several authors, give conclusion which shows different forms of results because all the studies are not done with same properties, section, admixture, salts, process and different factors for corrosion process. All information will be gathered from all the review which are added in this paper like factors, behavior, condition, corrosion description used by different authors. A full conclusion on the basis of all the data are as follows:

(1) Different types of set-up can be used for impressed current technique for artificial corrosion process, in which normally the current intensity varies in a range of 1 mA/cm^2 to 4 mA/cm^2. Normally 1%–4% NaCl or $CaCl_2$ water solution is used for better electrolysis process.

(2) Cover, cracks, water (carrying salt) and permeability are some of the major factors which are directly responsible for accelerating the corrosion process.

(3) Once cracks are formed, the rate of corrosion process is enhanced due to which bi-product of steel (rust) is increased which in turn increases the pressure due to change in volume thereby leading towards propagation of cracks.

(4) Pitting type of corrosion is found to be extensively hazardous than normal (general) corrosion, which ultimately leads to a sudden collapse of the structure.

(5) Approximately 50% loss in strength is observed in most of the past research studies. Several type of admixtures and coatings are used as a preventive measures for corrosion process.

(6) Due to corrosion in bar, cross-section of reinforcement as well as mass loss is observed approximately in a range of 15% to 30%.

(7) After load testing is performed, 0.05 mm to 15 mm cracks was observed for different specimens and in different failure zone (Flexure and shear zone).

(8) Findings also states that the chances of bond failure and reinforcement failure also increases due to corrosion mechanism in the concrete structures.

(9) Beams without corrosion depicted higher shear strength (up to 30%) than the corroded beam.

(10) Corrosion damage on steel is faster than the formation of cracks, as a result of which cracks appears to be dominant till the decrease in strength reaches up to 70%.

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Conflict of interest

All authors declare no conflicts of interest in this paper.

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