

Article

Effect of Reinforcement Corrosion on Structural Behavior in Reinforced Concrete Structures According to Initiation and Propagation Periods

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Abstract: Reinforced concrete (RC) structures lose performance over time. One of the most significant environmental factors causing performance degradation is reinforcement corrosion. Corrosion reduces the diameter of the reinforcement and degrades the mechanical properties of both the concrete and the steel. In order to determine the degradation of the structural performance caused by corrosion, it is essential to identify the time periods in which corrosion initiates and propagates, and to thoroughly analyze the parameters that govern these processes. In this study, the corrosion initiation and propagation times were calculated separately for an RC frame exposed to chloride ions. Factors contributing to the further reduction of these processes were identified. Analyses and comparisons were made to determine the structural performance of the calculated RC frame over its service life. The results showed that corrosion reduces the structural performance, especially in regions with high seismic potential, where preventive measures against corrosion are essential. The most important parameter influencing the corrosion mechanism is the corrosion rate, while another important factor is the formation of cracks on the element surface. In cases where cracks are present, the corrosion initiation time is shortened, which means that materials should be selected at the design stage that are suitable for the environment, and the years of maintenance required should be calculated by the design engineers. In addition, the surfaces of RC elements should be monitored, and crack tracking should be carried out.

Keywords: reinforcement corrosion; reinforced concrete structures; chloride ion exposure; service life analysis; corrosion initiation; corrosion propagation



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1. Introduction

Due to environmental influences, an electrochemical process occurs between the metal and its environment. As a result of the process, changes in the physical and chemical properties of the material are called corrosion [1]. With reinforcement corrosion occurring in RC structures, the reinforcement cross-section is reduced, mechanical properties change, and cracks and spalling occur in concrete. This situation adversely affects durability and structural performance. For this reason, possible corrosion scenarios should be taken into consideration during the project phase. Concrete protects reinforcement bars from external factors. Additionally, a very thin oxide layer forms around the reinforcement, which helps protect it from corrosion in a highly alkaline environment. The main factors causing the deterioration of the oxide layer are chloride ions or carbonation. The most significant factor is chloride ions [2]. The concentration of chloride ions that breaks the passivity protecting the reinforcement and initiates corrosion is referred to as the critical value. The time that passes until the passivity is broken is the initiation phase of corrosion. After the passivity is broken, the progression phase of corrosion begins. With the start of corrosion, a rust layer that expands in volume around the reinforcement begins to form. This rust layer causes

an increase in tensile forces, leading to cracking and the eventual spalling of the cover concrete, which has low tensile strength. The spalling of the cover concrete exposes the reinforcement to the atmosphere. This condition accelerates the rate of corrosion, reduces bond strength, and decreases the overall performance of the structure [3].

Corrosion clearly reduces the service life of structures. To mitigate this risk, reinforced concrete (RC) structures should be designed to resist environmental factors. For existing structures, assessing the initiation and progression of corrosion is essential to plan maintenance schedules and costs effectively. Numerous studies in the literature focus on determining the initiation and propagation phases of corrosion.

Kwon et al. [4] estimated the service life of RC structures subjected to marine environments by accounting for the impact of cracks induced by chloride ions, and validated their predictions against field data. They noted that durability damage increases with crack width, and, inversely, the service life decreases. Smilauer et al. [5] studied the initiation times of corrosion due to carbonation and chloride ions in cracked concrete structures. They developed the Papadakis model for carbonation and the Kwon model for chloride-induced corrosion and compared their findings with 10 years of field data. They stated that the presence of a 0.3 mm wide crack in the structure accelerates the initiation time of corrosion by five times. Kim Anh T. and Mark G. Stewart [3] developed models for chloride-induced corrosion in RC bridges in 2000 and investigated their structural reliability. Using the model developed by Stewart and Rosowsky to determine the initiation time of corrosion, they applied the formula proposed by Liu and Weyers to reduce the diameter of the reinforcement. In their study, they proposed a life-cycle reliability model for structural failure. They noted that the thickness of the cover concrete and the water/cement (W/C) ratio have a significant impact on the probabilities of collapse. Ahmad [6] explained the electrochemical process involved in the formation of corrosion and analyzed models for predicting service life, including those developed by Morinaga, Bazant, Wang and Zhao, and Dagher and Kalendran. In his analysis, the initiation phase of corrosion was assumed to be zero. Ranjith et al. [7] conducted a parametric study to estimate the service life of corroded RC structures. They examined three bridge barriers, assuming the initiation phase of corrosion to be zero and calculating only the propagation period using the Morinaga, Bazant, Wang and Zhao, and IRC models for comparison. They found that the IRC model predicted a very low service life, noted that Bazant's model did not account for ribbing, highlighted that actual chloride values in the field could be higher, and suggested that these methods could be used to estimate repair timing in RC structures. Borah et al. [8] investigated the service life and the parameters affecting the service life of RC bridges influenced by chloride ions in India. They chose the Duracrete and Morinaga models as prediction methods and noted that these models produced results consistent with actual field data and could be applied to RC structures. Luping and Utgennant, however, pointed out that the Duracrete model's diffusion approach decreases over the long term, leading to insufficient service life estimations [9]. Çolakoğlu and Öztemel [10] investigated the linear and nonlinear behavior of RC columns under different corrosion damage levels across five corrosion scenarios. They examined the changes in an RC column with a propagation period of 15 years under corrosion rates of 0.15, 0.8, and 4, using the ANSYS software. Yüksel [11] examined the effects of reinforcement corrosion on the structural behavior of RC buildings. He conducted dynamic analyses by applying nine earthquake records to frames with different propagation periods and corrosion rates. Sakcalı and Yüksel [12] investigated corrosion damage in RC structures. They created two different corrosion rates and three different corrosion propagation periods, conducting various analyses on a building. They compared the changes in materials and sections, noting that the increase in corrosion levels reduced the strength of the cover concrete, particularly due to cracking and softening in the concrete. Carpio et al. [13] evaluated the flexo-compressive strength capacity of deteriorated RC elements according to its structural age, considering the effects of cracking in the cover and environmental aggressiveness. They found that cracked frame elements reduced their serviceability life by up to 62%, and the corrosion effects reduced

the element strength by up to 44% in 25 years. Additionally, considering the corrosion deterioration can prevent strength overestimations up to 1.46 times. The corrosion level has a significant impact on the strength and durability of RC elements. The crack width effect reduces the flexo-compressive strength up to 38% more, even within the recommended limit. The deterioration effects can be reduced by up to 55% by incrementing the rebar diameter or the cover width. So, they concluded that the corrosion level has a significant impact on the strength and durability of RC elements, and that the crack width effect is a critical factor in determining the serviceability life of these elements. Jung et al. [14] proposed a practical methodology to evaluate the seismic performance of RC structures with corrosion-damaged members. This methodology aims to estimate structural performance degradation factors based on the strength and deformability of corrosion-damaged members. There are numerous studies in the literature on the control and monitoring of corrosion damage; Croce et al. [15] showed that the degradation conditions and steel and concrete classes significantly affect the time-dependent reliability curves. Duvnjak et al. [16] found that changes in natural frequencies and modal damping coefficients can be used to detect corrosion damage, with modal damping being a more sensitive parameter. The potential of mode shapes in determining damage caused by corrosion. The corrosion resistance of different materials has also been investigated. For example, Wu et al. [17] evaluated that the corrosive environment of soda residue soil (SRS) has a significant influence on the deterioration of RC structures. Measures such as improving concrete strength, reducing the water–binder ratio, and adding mineral admixtures and fiber-reinforcing agents can enhance the durability of RC structures in SRS environments. Maidi et al. [18] concluded that structures reinforced with carbon fiber reinforced polymer (CFRP) have greater shear capacity and slower degradation rates than those reinforced with steel, and that CFRP shows promise in corrosive environments. Granata [19] found that the use of different techniques such as steel jacketing, fiber reinforced polymer (FRP) plates, and steel–concrete composite floors for strengthening and passive seismic protection strategy can allow engineers to obtain the result of structural adaptation to earthquakes with low-cost interventions.

As can be seen, most studies in the literature focus on separately calculating the initiation time, the propagation time, or the behavior of buildings at a given age. However, no comprehensive model exists that integrates all these aspects for the same RC building. In this study, the corrosion initiation and propagation times for an existing RC frame were calculated using different methods, and the structural performance for these years as a whole was determined using actual earthquake records. The corrosion effect parameters were applied to the frame according to its service life and dynamic analyses were performed for each scenario. The structural performance of an uncorroded frame was compared with frames of different service lives and the parameters affecting the structural performance were investigated. This study will provide valuable information on the evaluation periods of RC buildings and will make a significant contribution to the literature for future research.

2. Structure Service Life

The service life is defined as the time period from the construction of the structure until it requires maintenance [20]. Tuutti [21] divided the service life into two phases: the corrosion initiation (t_i) and corrosion propagation (t_p) stages. The sum of these two phases gives the time (t) at which the structure requires repair (Figure 1).

Different researchers have developed various models to predict these durations, and below, the prediction processes using the Kwon and Morinaga Models, which were preferred in this study, are explained.

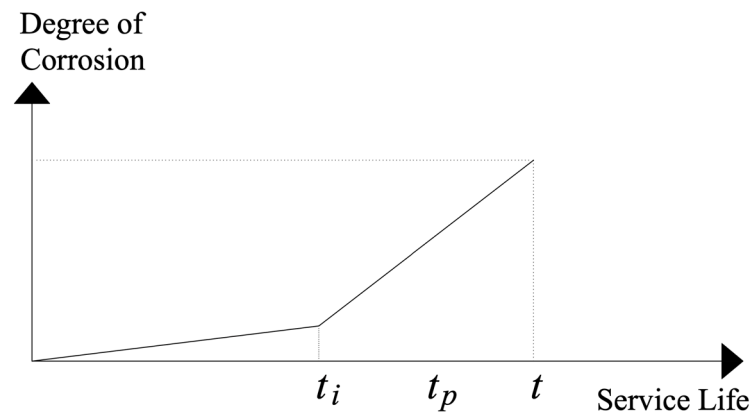


Figure 1. Service life of a structure affected by corrosion caused by chloride ions.

2.1. Corrosion Initiation Time

When chloride ions reach the critical level, the passive layer around the reinforcement is broken, and corrosion begins (t_i). The corrosion initiation time varies depending on the cover concrete thickness and the concentration of Cl^- ions. The rate at which Cl^- ions penetrate the concrete depends on the concrete composition and environmental conditions. Various corrosion initiation time prediction models have been studied in the literature [4,22–24].

Chloride ions accumulated on the concrete surface are transported into the concrete through various mechanisms; environmental factors and the moisture condition of the concrete also influence how the ions penetrate the concrete. The ions pass through the cover concrete and move toward the reinforcement. Corrosion starts when the amount of chloride on the surface of the reinforcement surpasses the critical chloride concentration level. The estimation of this duration is based on Fick's second law of diffusion [25].

$$\frac{\partial C(x,t)}{\partial t} = D \frac{\partial^2 C(x,t)}{\partial x^2} \quad (1)$$

$C(x,t)$ is the chlorine concentration at depth x at time t , and D is the diffusion coefficient. In the model developed by Kwon, $C(x,t)$ chlorine concentration is calculated by Equation (2) [4].

$$C(x,t) = C_s \left\{ 1 - \operatorname{erf} \left(\frac{x}{2\sqrt{D_m(t)f(w)t}} \right) \right\} \quad (2)$$

In the equation, C_s represents the chloride concentration at the surface (kg/m^3), erf is the error function, x is the distance from the surface to the point where chloride is measured (mm), D_m is the diffusion coefficient at the time of calculation (mm^2/s), $f(w)$ is the scaling function for cracks formed in the concrete due to corrosion, and t represents time. The diffusion coefficient varies depending on the concrete's water–cement ratio, temperature, type of binder, and its amount. Additionally, Ju et al. [26] found that coarse aggregate content and particle grading significantly affect chloride diffusion behavior in concrete, too. The diffusion coefficient at time t is calculated using Equation (3).

$$D(t) = D_{ref} \left(\frac{t_{ref}}{t} \right)^m \quad (3)$$

Here, m is a constant depending on mix proportions of concrete; it is 0.37 for Portland cement [8]. D_{ref} is the diffusion coefficient at the reference time, typically considered to be 28 days. The average diffusion coefficient, $D_m(t)$, is calculated using Equation (4a,b) [27].

$$D_m(t) = \frac{D_{ref}}{1-m} \left(\frac{t_{ref}}{t} \right)^m, \quad t < t_R, \quad (4a)$$

$$D_m(t) = D_{ref} \left[1 + \frac{t_R}{t} \left(\frac{m}{1-m} \right) \right] \left(\frac{t_{ref}}{t} \right)^m, \quad t \geq t_R \quad (4b)$$

In the equation, t_{ref} represents the time at which the diffusion coefficient is measured, and the parameter t_R is typically taken as 30 years [16].

In the case of cracks in the concrete, the chloride concentration increases, which leads to an increase in the average diffusion coefficient $D_m(t)$. Therefore, the scaling function used to account for crack formation in the concrete is calculated using Equation (5) [4].

$$f(w) = 31.61w^2 + 4.73w + 1 \quad (5)$$

In the equation, w represents the crack width in millimeters. An increase in crack width will lead to an increase in this ratio as well.

2.2. Corrosion Propagation Time

After corrosion begins, the time until the corrosion products, which expand in volume around the reinforcement, cause cracking of the cover concrete is referred to as the corrosion propagation phase (t_p). Researchers have developed various prediction models to determine this duration [6,7,20,22].

Morinaga [13] developed Equation (6) by relating the propagation time to the corrosion rate, cover thickness, and reinforcement diameter.

$$Q_{cr} = 0.602d \left(1 + \frac{2C}{d} \right)^{0.85} \quad (6)$$

Here, Q_{cr} is the amount of corrosion that causes cracking in the concrete ($\times 10^{-4}$ g/cm²), d is the reinforcement diameter (mm), and C is the cover thickness (mm). The corrosion propagation time (t_p) is calculated using Equation (7).

$$t_p = \frac{Q_{cr}}{j_r} \quad (7)$$

In the equation, j_r is the instantaneous corrosion rate (g/cm²s), and it is calculated using Equation (8).

$$j_r = \frac{W}{F} i_{corr} \quad (8)$$

Here, W is the equivalent weight of steel (27,925 g), F is the Faraday constant (96,485 C), and i_{corr} is the corrosion current density (μ A/cm²). The corrosion current density varies according to different researchers. In Morinaga's model, the corrosion current density ranges from 0.1 to 6 (μ A/cm²) according to Bazant [22]. Vu-Stewart [3] calculated the corrosion current density based on the water–cement ratio (Equation (9)). The corrosion levels accepted by some researchers are also shown in Table 1.

$$i_{corr}(1) = \frac{37.8 (1 - w/c)^{-1.64}}{c} \quad (\mu\text{A}/\text{cm}^2) \quad (9)$$

Table 1. Corrosion rate classification based on corrosion current density [10,28].

Corrosion Classification	Corrosion Rate ($\mu\text{A}/\text{cm}^2$)		
	Dhir et al. (1994)	Brite/Euram (1995)	Middleton and Hogg (1998)
Corrosion free	-	<0.1	-
Low	0.1	0.1–0.5	0.1–0.2
Middle	1	0.5–1.0	0.2–1.0
High	10	>1.0	>1.0

Borah et al. calculated the corrosion propagation time of a RC bridge pier using the Morinaga model and stated that this method can be used for calculating the corrosion propagation times of RC structures [8]. This method was also preferred in this study.

3. Numerical Modelling

In the study, a 4-span, 8-story frame has been selected to assess the condition of existing RC buildings. Considering that the frame is on the seaside, it is assumed that all elements on each floor are affected by wind-blown seawater. The RC columns, beams, and the frame system constituting the structure are shown in Figure 2.

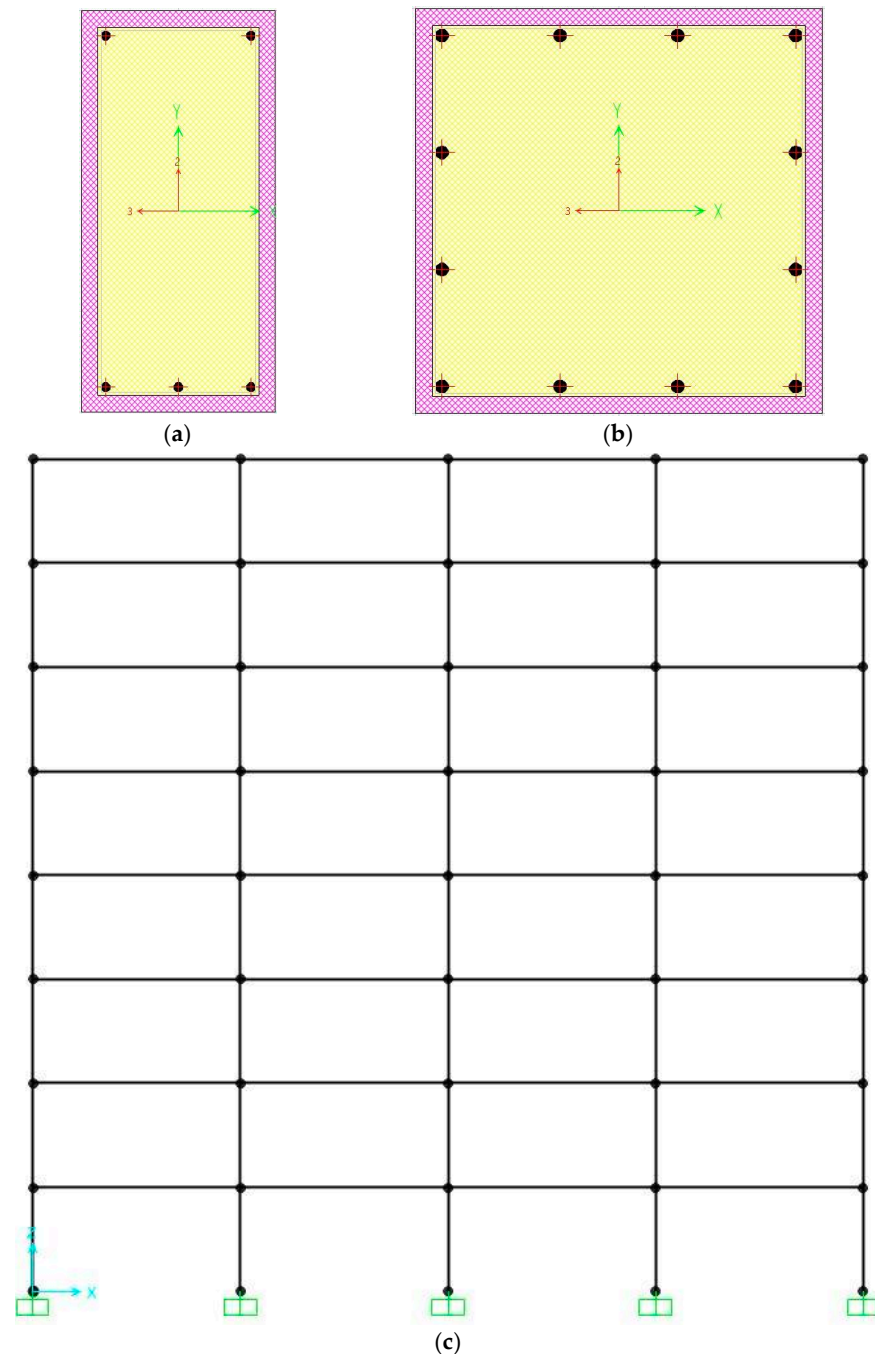


Figure 2. RC beam (a), column (b), and frame system (c).

The dynamic analysis of the frames, where fiber hinges are used for columns and beams, is performed using the SAP2000 [29] structural analysis program. There are two pri-

many methods for modeling the nonlinear behavior of structural system elements: lumped plasticity and distributed plasticity (Figure 3). The distributed plasticity method inherently considers the moment–rotation relationship and the plastic axial stress for each fiber, offering greater accuracy compared to the lumped plasticity approach [30–32]. In this study, nonlinear fiber hinges, a type of distributed plasticity model illustrated in Figure 3c, have been utilized. The fiber hinge lengths at the ends of columns and beams are defined as half the height of the cross-section in the respective direction of the structural element [3,31].

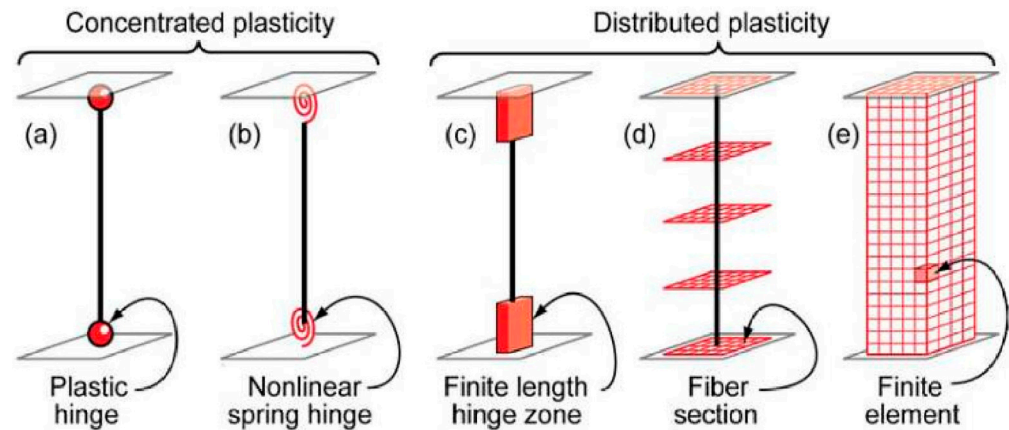


Figure 3. Idealized nonlinear models for beam and column elements; (a) plastic hinge, (b) nonlinear spring hinge, (c) finite length hinge zone, (d) fiber section, (e) finite element [30].

The damage conditions related to the stresses in both the concrete and reinforcement of the columns and beams are evaluated according to the acceptance criteria calculated based on TBEC-2018 [31]. The rotation values required to determine the IO (immediate occupancy), LS (life safety), and CP (collapse prevention) states for the columns and beams are defined according to TBEC-2018. The rotation values obtained from the analysis are compared with those defined in the regulation. The bending stiffness used in the dynamic analysis is calculated based on the effective section stiffness factors specified in TBEC-2018, which are 0.70 for columns and 0.35 for beams [3,31,33–36]. The earthquake record shown in Figure 4 is used in the dynamic analysis and applied in the X direction to the frame.

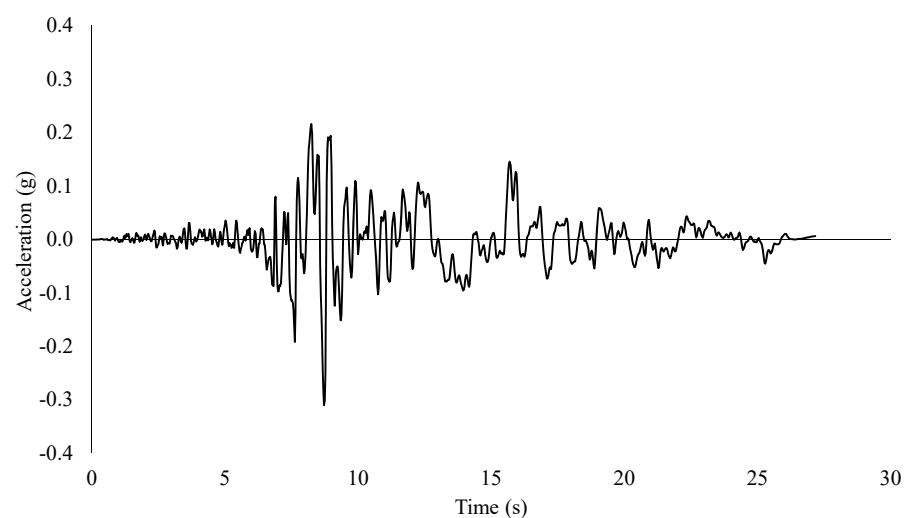


Figure 4. Earthquake record.

The building's distance from the sea has been taken as 300 m. There is very little actual field data available from coastal areas [3]. Based on McGee's study of 1158 bridges and their distance from the sea, since 300 m is smaller than 2.84 km, the surface chloride

concentration has been assumed to be 2.415 kg/m^3 . The building has been considered at the calculated ages for the service life. Concrete strength C20, a W/C ratio of 0.40, cement class CEMI Portland cement, and steel class B420C have been assumed. The chloride threshold value has been taken as 0.05% [1]. According to TS500 [37], the allowable crack limit for aggressive conditions inside and outside the structure has been accepted as a crack value of 0.1 mm for elements.

The corrosion initiation times for cracked and non-cracked conditions were calculated using the Kwon Model, based on the frame and environmental properties, and are shown in Table 2.

Table 2. Corrosion initiation times.

Crack Situation	Initiation Time (Year)
The frame without crack	12
The frame with crack	5

The propagation time was calculated using the Morinaga model. For all corrosion levels, common values of $0.5 \text{ } (\mu\text{A/cm}^2)$, $1 \text{ } (\mu\text{A/cm}^2)$, and $6 \text{ } (\mu\text{A/cm}^2)$ current density were used. Additionally, a corrosion current density of $3.495 \text{ } (\mu\text{A/cm}^2)$ calculated based on the W/C ratio was also included in the study (Equation (9)). Accordingly, propagation times were calculated using four different corrosion rates, and Table 3 was created.

Table 3. Corrosion propagation times.

Corrosion Rate ($\mu\text{A/cm}^2$)	Propagation Time (Year)
0.5	22
1	11
3.495	3
6	2

The corrosion service lifetimes calculated based on the corrosion initiation and propagation times are shown in Table 4. Here, two different service durations were calculated and evaluated based on whether cracks were present or not in the frame.

Table 4. The change in corrosion initiation time, propagation time and service life of the frames according to different corrosion rates and cracked/non-cracked conditions.

Crack Situation	Corrosion Rate ($\mu\text{A/cm}^2$)	Initiation Time (Year)	Propagation Time (Year)	Service Life (Year)
The frame without crack	0.5	12	22	34
	1	12	11	23
	3.495	12	3	15
	6	12	2	14
The frame with crack	0.5	5	22	27
	1	5	11	16
	3.495	5	3	8
	6	5	2	7

As seen, the corrosion initiation time is independent of the corrosion rate and changes only based on the concrete and environmental factors, while the propagation time is affected by the corrosion rate. The presence of cracks in the section reduces the service life. RC buildings are constructed with a service life of 50 years. Therefore, the condition of the frame after 50 years with the same corrosion rate and corrosion initiation time is also included in the study. Accordingly, the models and characteristics to be examined in the numerical modeling are shown in Table 5. The calculated ages of the RC frame are 15, 23, 24, 34, and 50 years.

Table 5. Models for service life calculation based on corrosion rates and corrosion propagation durations.

The Model Number	The Model Code	Corrosion Rate ($\mu\text{A}/\text{cm}^2$)	Propagation Time (Year)	Service Life with Crack (Year)	Service Life Without Crack (Year)
MREF	MREF	0	0	0	0
M1	M1-0.5-22	0.5	22	27	34
M2	M2-1-11	1	11	16	23
M3	M3-3.5-3	3.495	3	8	15
M4	M4-6-2	6	2	7	14
M5	M5-0.5-38	0.5	38	-	50
M6	M6-1-38	1	38	-	50
M7	M7-3.5-38	3.495	38	-	50
M8	M8-6-38	6	38	-	50
M9	M9-0.5-45	0.5	45	50	-
M10	M10-1-45	1	45	50	-
M11	M11-3.5-45	3.495	45	50	-
M12	M12-6-45	6	45	50	-

The name “M1-0.5-22” refers to a corrosion rate of 0.5, and a corrosion propagation duration of 22 years. M1 refers to the model number. All models have been named according to this rule. The models were created to examine the condition of the frame at the moment it reaches its service life. The evaluations were conducted based on propagation durations. In total, 12 solutions and 1 reference model (no corrosion) were created, resulting in 13 separate models. The corrosion effects considered in the modeling include reductions in rebar diameter, rebar mechanical properties, and concrete mechanical properties. The reduction in rebar diameter is calculated using Equation (10).

$$\Phi(t) = \Phi_0 - 2 P_x = \Phi_0 - 2i_{corr} k (t - t_{in}) \quad (10)$$

Here, $\Phi(t)$ represents the rebar diameter at time t (mm), Φ_0 is the nominal rebar diameter (mm), i_{corr} is the corrosion current density ($\mu\text{A}/\text{cm}^2$), t_{in} is the corrosion initiation time (years), and P_x is the average processing value of the attack (mm). The coefficient k is taken as 0.0116, which is used to convert the corrosion rate from $\mu\text{A}/\text{cm}^2$ to mm/year. P_x , the average processing value, is defined by the loss in rebar diameter [38,39].

Corrosion alters the mechanical properties of the reinforcement [40]. To determine this change, the mass loss (Δ_w) in the reinforcement is first calculated using Equation (11).

$$\Delta_w = ((D_i^2 - D^2)/D_i^2) \times 100 \quad (11)$$

D_i represents the initial diameter, and D represents the remaining diameter after corrosion. According to the experimental data of Lee Cho [41], equations have been developed to calculate the yield strength (σ_y), tensile strength (σ_u), modulus of elasticity (E), and elongation rate (δ) of the steel. Since their study focused on steel similar to B420C steel, these equations have been used in the present study. In the equations, the index “ i ” refers to the first property.

$$\sigma_y = (1 - 1.24(\Delta_w/100)) \cdot \sigma_{yi} \quad (12)$$

$$\sigma_u = (1 - 1.07(\Delta_w/100)) \cdot \sigma_{ui} \quad (13)$$

$$E = (1 - 0.75(\Delta_w/100)) \cdot E_i \quad (14)$$

$$\delta = (1 - 1.95(\Delta_w/100)) \cdot \delta_i \quad (15)$$

The remaining reinforcement diameters and mass losses due to corrosion at the end of the service life of the columns in the models are shown in Table 6.

Table 6. Reinforcement diameter and mass losses due to corrosion.

Model Code	Nominal Diameter (mm)	Dia After Cor. (mm)	Mass Loss (%)	Model Code	Nominal Diameter (mm)	Dia After Cor. (mm)	Mass Loss (%)
M1-0.5-22	8	7.75	6.28	M7-3.5-38	8	4.92	62.20
	12	11.75	4.21		12	8.92	44.76
	16	15.75	3.16		16	12.92	34.81
M2-1-11	8	7.75	6.28	M8-6-38	8	2.71	88.52
	12	11.75	4.21		12	6.71	68.73
	16	15.75	3.16		16	10.71	55.19
M3-3.5-3	8	7.76	5.99	M9-0.5-45	8	7.48	12.62
	12	11.76	4.01		12	11.48	8.51
	16	15.76	3.02		16	15.48	6.42
M4-6-2	8	7.72	6.84	M10-1-45	8	6.96	24.40
	12	11.72	4.59		12	10.96	16.64
	16	15.72	3.45		16	14.96	12.62
M5-0.5-38	8	7.56	10.72	M11-3.5-45	8	4.35	70.42
	12	11.56	7.21		12	8.35	51.57
	16	15.56	5.43		16	12.35	40.41
M6-1-38	8	7.12	20.83	M12-6-45	8	1.74	95.29
	12	11.12	14.15		12	5.74	77.15
	16	15.12	10.72		16	9.74	62.97

The deterioration in concrete due to corrosion, represented by the softening coefficient (ζ), has been calculated using Equation (16) based on the work of Hsu et al. [42].

$$\zeta = \frac{0.9}{\sqrt{1 + 600\varepsilon_r}} \quad (16)$$

In the equation, ε_r represents the lateral reinforcement deformation.

4. Analysis Results and Discussions

4.1. Base Shear, Floor Displacement and Top Displacement

In the study, dynamic analyses were performed, resulting in base shear force, peak displacement, and story displacement data. The peak displacement and base reaction values of the frame obtained at the end of the analysis are shown in Figure 5. The maximum base shear force occurred in the reference frame, with a value of 329.87 kN and a displacement of 42.11 mm. With the emergence of corrosion-related effects, it is observed that the base reaction decreased, while the peak displacement increased. Additionally, when displacements are examined by story levels, it is seen that as the story height increases, the displacement of the frame also increases (Figure 6). Changes were observed to appear in two separate regions. Models M7, M8, M11, and M12 show more displacement and reduced base shear force compared to other models. The corrosion propagation durations of these frames are 38 and 45 years, with corrosion rates of 3.5 and 6 $\mu\text{A}/\text{cm}^2$, respectively. Therefore, the main cause of the weakening in the system is the 3.5 to 6 times increase in corrosion rate compared to other models. For this reason, the corrosion rate should be examined in detail. Comparisons of base shear force and peak displacement due to mass loss are presented in Figures 7 and 8, respectively.

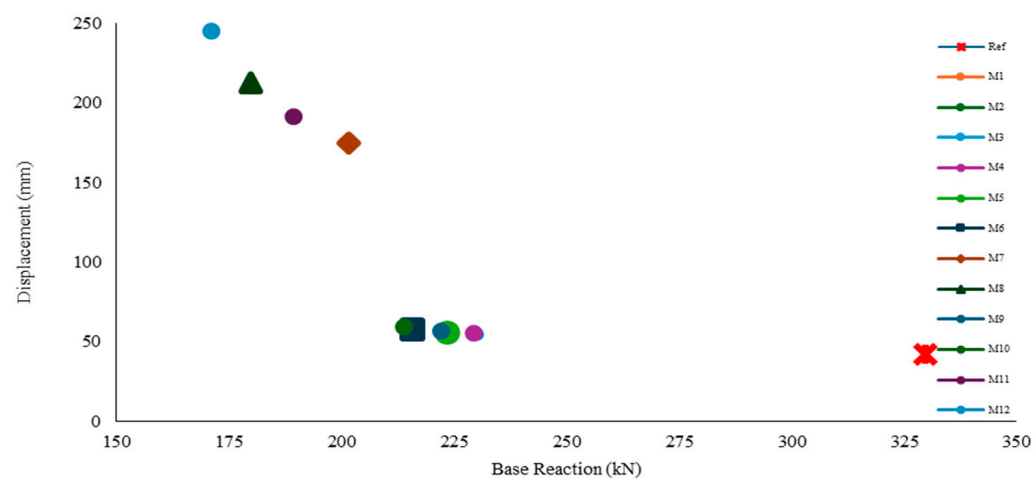


Figure 5. Base reactions and displacement values of the models.

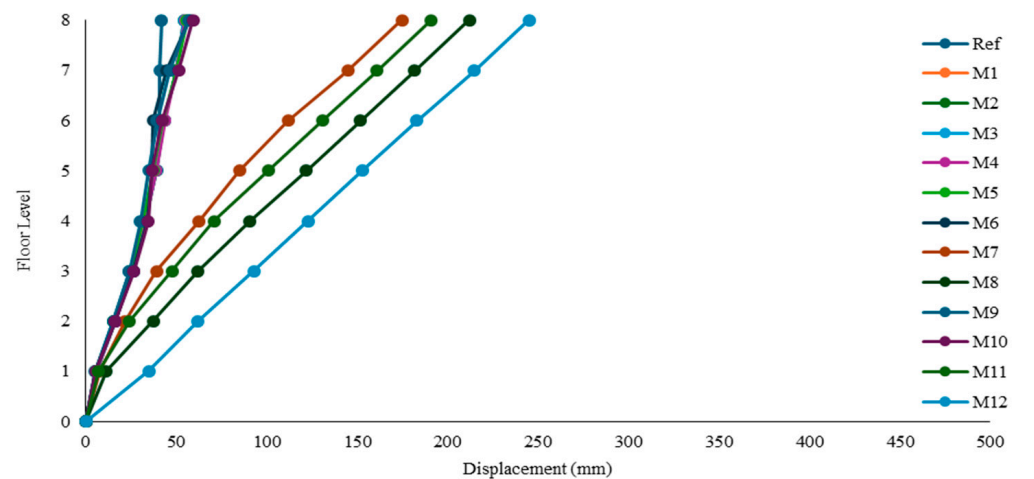


Figure 6. Displacement values at floor level.

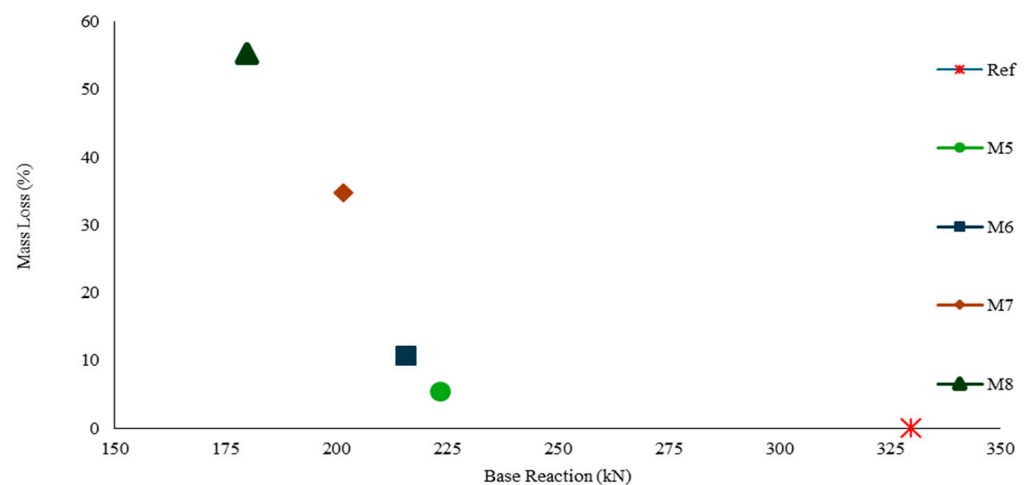


Figure 7. The effect of corrosion rate on mass loss and base reaction.

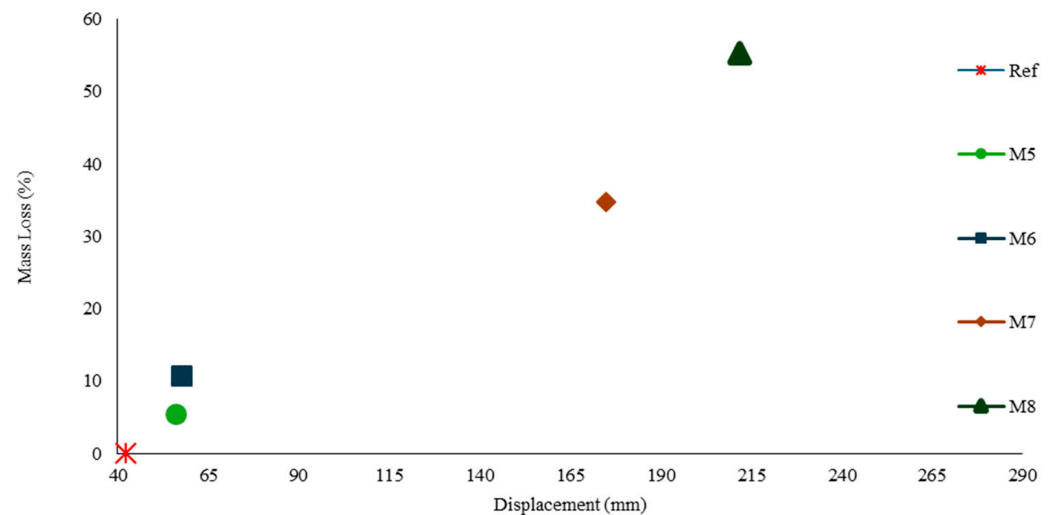


Figure 8. The effect of corrosion rate on mass loss and displacement.

The M5–M8 models with a 38-year service life were examined to investigate the effects of mass loss in the longitudinal reinforcement ($\Phi 16$) on base reaction and displacement. The maximum base reaction of 329.92 kN occurred in the reference model with no corrosion, while the lowest base reaction, after a 5.43% mass loss, decreased by 32.3%. It is observed that an increase in mass loss in the reinforcement causes significant reductions in base reaction. Conversely, a similar trend was observed for displacement. The lowest displacement of 42.11 mm occurred in the reference model. As the mass loss increased by 55.19%, the displacement increased fivefold. Since mass loss in the reinforcement and degradation of mechanical properties in both the reinforcement and concrete weakened the frame, the frame experienced large displacements even under smaller shear forces. Here, it is evident that corrosion rate has an impact on both base reaction and displacement.

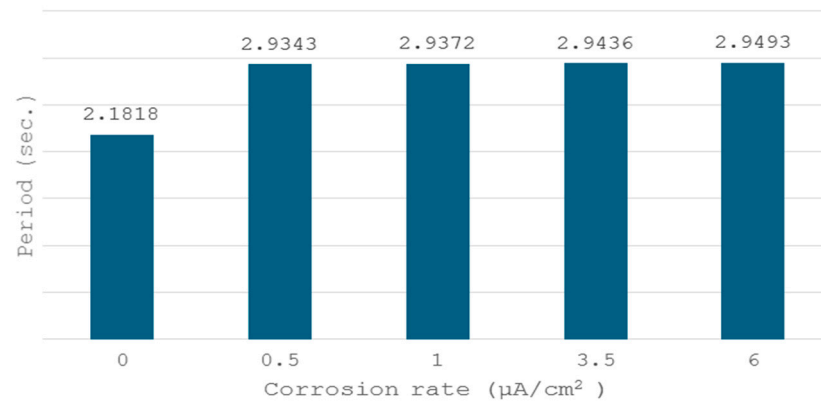
4.2. Period–Mass Loss Relationship

The main parameter determining the effects of corrosion is mass loss, and it has been explained above that all variables are calculated based on this loss. Mass loss is also calculated according to the reinforcement diameter (Equation (11)). Therefore, to make evaluations based on mass loss, different reinforcement diameters should be examined separately. At the same corrosion rate, mass loss in larger diameter reinforcements is smaller, while mass loss increases as the reinforcement diameter decreases. This situation is clearly shown in Table 7 above. Table 7 below shows the period of the frame in the analyses. The period of a structure refers to the time it takes for the structure to oscillate at its natural frequency during an earthquake. Rigid structures have shorter periods. The weaker the structure, the more it oscillates, and the longer the period. In light of all this information, when examining Table 7, the lowest period occurs in the reference model, where no corrosion is present, at 2.1818 s, while the highest period is observed in the M12 model, which has the highest corrosion rate and propagation time, at 2.9494 s. The period has increased by 35.2% due to the corrosion weakening the system.

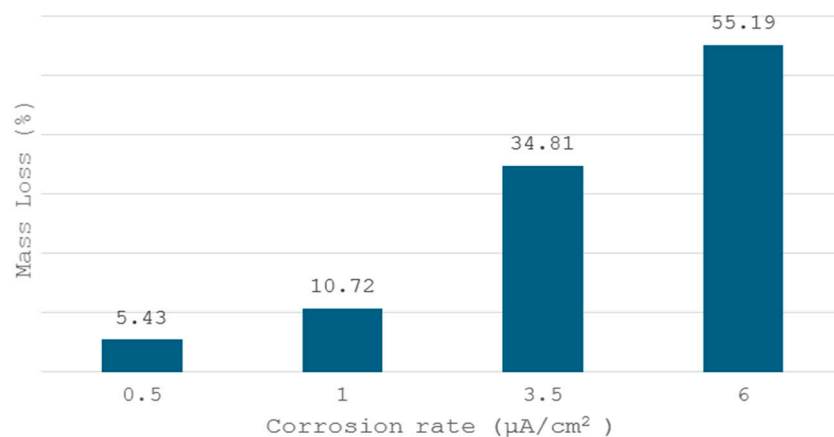
To examine the factors affecting the periods, the models should be compared within themselves. The comparison of the M5–M8 models with the same corrosion propagation time of 38 years, based on corrosion rates, is shown in Figure 9. According to this, it is observed that, as the corrosion rate increases, the period also increases. With the onset of corrosion, the period of the frame increased by 34.5%. An increase in the corrosion rate from $0.5 \mu\text{A}/\text{cm}^2$ to $1 \mu\text{A}/\text{cm}^2$ resulted in a 10% increase in the period, while an increase from $1 \mu\text{A}/\text{cm}^2$ to $3.5 \mu\text{A}/\text{cm}^2$ caused a 22% increase, and from $3.5 \mu\text{A}/\text{cm}^2$ to $6 \mu\text{A}/\text{cm}^2$, the period increased by 20%.

Table 7. The period values based on the mass loss.

Model Code	Mass Loss of $\varnothing 16$ (%)	Mass Loss of $\varnothing 12$ (%)	Mass Loss of $\varnothing 8$ (%)	Period (s)
MREF	0	0	0	2.1818
M1-0.5-22	3.16	4.21	6.28	2.9337
M2-1-11	3.16	4.21	6.28	2.9337
M3-3.5-3	3.02	4.01	5.99	2.9337
M4-6-2	3.45	4.59	6.84	2.934
M5-0.5-38	5.43	7.21	10.72	2.9343
M6-1-38	10.72	14.15	20.83	2.9372
M7-3.5-38	34.81	44.76	62.20	2.9436
M8-6-38	55.19	68.73	88.52	2.9493
M9-0.5-45	6.42	8.51	12.62	2.9344
M10-1-45	12.62	16.64	24.40	2.9372
M11-3.5-45	40.41	51.57	70.42	2.9446
M12-6-45	62.97	77.15	95.29	2.9494

**Figure 9.** Comparison of models with a corrosion propagation time of 38 years in terms of corrosion rates and periods.

It was previously mentioned that mass loss changes with corrosion rate, and this change varies depending on the diameters. Similarly, the comparison of mass losses and corrosion rates in the 38-year service life of $\varnothing 16$ longitudinal reinforcement is shown in Figure 10. As the corrosion rate increases, the mass losses also increase. With the corrosion rate rising from $0.5 \mu\text{A}/\text{cm}^2$ to $6 \mu\text{A}/\text{cm}^2$, the loss in reinforcement with a diameter of 16 mm increases from 5.43% to 55.19%.

**Figure 10.** Comparison of models with a corrosion propagation time of 38 years in terms of corrosion rates and mass loss.

The change in periods due to mass loss was compared between the models with a propagation time of 38 years (Figure 11). The period of the reference frame without any corrosion was 2.1818 s. With the occurrence of corrosion, the diameter of the reinforcements started to decrease, and the material started to deteriorate, which caused the period to increase. The period of the frame with the smallest mass loss (5.43%) increased by 34.5%, while the period of the frame with the maximum mass loss (55.19%) increased by 35.2%. This situation shows that the most significant increase in the period starts with the emergence of corrosion, but with the progression of corrosion and the increase in mass loss, the change in the period does not increase at the same rate and increases the period by small amounts.

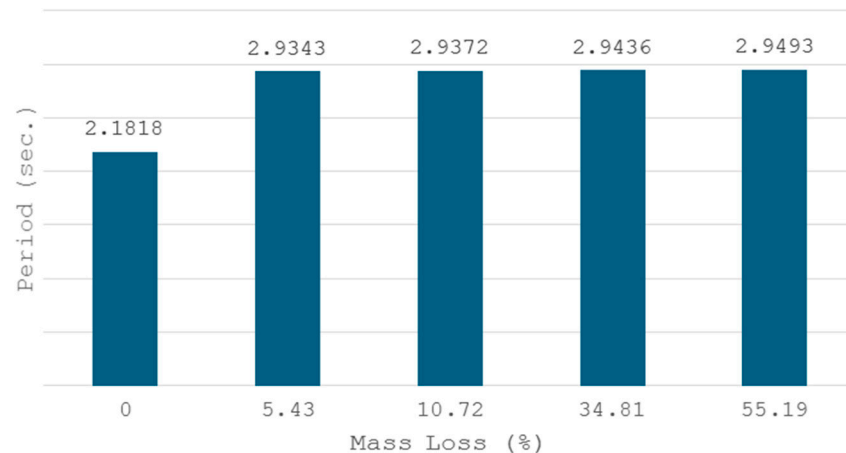


Figure 11. Comparison of models with a corrosion propagation period of 38 years in terms of mass loss and periods.

To investigate the effects of corrosion propagation times on mass loss and period, frames with the same corrosion rates and propagation times of 38 and 45 years were compared (Table 8). For frames with the lowest corrosion rate of $0.5 \mu\text{A}/\text{cm}^2$, increasing the corrosion propagation time by seven years increased the mass loss in $\Phi 16$ reinforcement by 18.2% and increased the period by 0.003%. In the frame with a corrosion rate of $6 \mu\text{A}/\text{cm}^2$, an increase in the propagation time of seven years increased the mass loss in $\Phi 16$ reinforcement by 14.1% and increased the period by 0.003%. The increase in the propagation time did not cause a significant change in the period. It is seen that the change in the period of the frames at different corrosion rates with the same propagation times is the same. It also caused a small change in mass loss. It is seen that the main difference is the corrosion rate.

Table 8. The period values based on the mass loss and corrosion propagation time.

Model Code	Corrosion Rate ($\mu\text{A}/\text{cm}^2$)	Propagation Time (Year)	Mass Loss of $\Phi 16$ (%)	Period (s)
M5-0.5-38	0.5	38	5.43	2.9343
M9-0.5-45	0.5	45	6.42	2.9344
M8-6-38	6	38	55.19	2.9493
M12-6-45	6	45	62.97	2.9494

4.3. Moment–Curvature

Moment–curvature graphs for all models were drawn for the columns located on the ground floor, which have the highest axial load (Figure 12). Accordingly, the non-corroded reference model has the highest moment capacity at 504 kNm. When the maximum moment capacity was reached, a curvature of 0.0172 1/m occurred in the section, after which the moment began to decrease, and the curvature increased. In the reference model, the section

reached a curvature of 0.04 1/m by the end of the analysis. In the M1 model, where corrosion occurred at the lowest rate, the maximum moment decreased by 44.5% and the maximum curvature by 23.75%. The M12 model, which has the most severe corrosion effects, had the lowest moment capacity among the models, falling to 82.7% of the reference model and 75% below in terms of curvature. The M12 model reached its maximum moment at 57.6% less curvature compared to the non-corroded model. In general, corrosion reduced both the moment capacity and curvature of the section. In other words, as the frame weakened, permanent deformations in the section increased. The main factor causing the reduction in moment and curvature was the mass loss in the reinforcements and the resulting deterioration in mechanical properties. According to Table 9, it can also be seen that, with an increase in mass loss, moment and curvature decreased. In the M3 model, where the mass loss was 3.02%, the maximum moment decreased by 30.8%. In the M5 and M9 models, where the corrosion rate was kept constant and the propagation time varied, a seven-year increase in propagation time increased the mass loss as previously explained, and the maximum moment decreased by 2.3%. In the M5 and M7 models, where the propagation time was kept constant and the corrosion rate varied, the moment decreased by 51.5%. It is clearly seen that the parameter effective in the moment-carrying capacity of the section is again the corrosion rate.

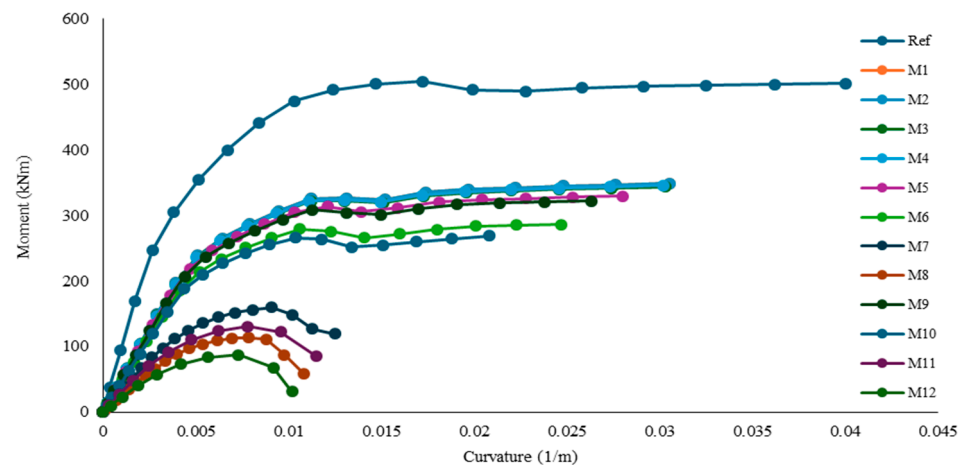


Figure 12. Moment–curvature relationship of ground floor columns.

Table 9. Moment–curvature and mass loss data for ground floor columns.

Model Code	Mass Loss of Ø16 (%)	Maximum Moment (kNm)	Maximum Curvature (1/m)
MREF	0	504.14	0.04
M1-0.5-22	3.16	345.38	0.0303
M2-1-11	3.16	345.38	0.0303
M3-3.5-3	3.02	349.03	0.0305
M4-6-2	3.45	343.45	0.0302
M5-0.5-38	5.43	329.48	0.028
M6-1-38	10.72	285.75	0.0247
M7-3.5-38	34.81	159.74	0.0125
M8-6-38	55.19	113.49	0.0108
M9-0.5-45	6.42	321.89	0.0263
M10-1-45	12.62	268.56	0.0208
M11-3.5-45	40.41	130.80	0.0115
M12-6-45	62.97	87.29	0.0102

4.4. Element Damage Levels

Hinge conditions under the effect of an earthquake provide important information about the structural performance. The frame was evaluated based on the actual earthquake

load determined for each model (Figure 13). The damage levels of columns and beams in the IO (immediate occupancy), LS (life safety), and CP (collapse prevention) states were assessed according to TBEC-2018 [30–36]. It is observed that, as the deteriorating effects of corrosion increase, the damage level in the columns and beams advances. While all beams and columns were between IO–LS in the reference model, even the initiation of corrosion at the lowest rate increased the damage in all beams, pushing it into the LS–CP range. Since columns are stronger than beams, the damage in columns did not progress as much as in beams. With the increase in the corrosion propagation time from 22 to 38 years, 25% of the beams advanced to the CP damage state. When the corrosion rate increased from $0.5 \mu\text{A}/\text{cm}^2$ to $6 \mu\text{A}/\text{cm}^2$ within a 38-year service life, the percentage of beams at the CP damage level increased from 25% to 87.5%. This indicates that the increase in corrosion rate has a significant effect on the damage levels in beams. The situation in columns was different from that in beams. In the M8 frame, where the corrosion rate was $6 \mu\text{A}/\text{cm}^2$ and the propagation time was 38 years, 37.5% of the columns advanced to the LS–CP damage state. In the models before this, the damage level remained between IO–LS. Additionally, in the M12 model, which had the most aggressive corrosion among the models, collapse-level damage was observed in a column for the first time. In the M8 and M12 models, which had the same corrosion rate, the extension of the service life from 38 to 45 years advanced the damage level in 32.5% of the columns at the IO–LS level, with 7.5% progressing to LS–CP and 37.5% to CP.

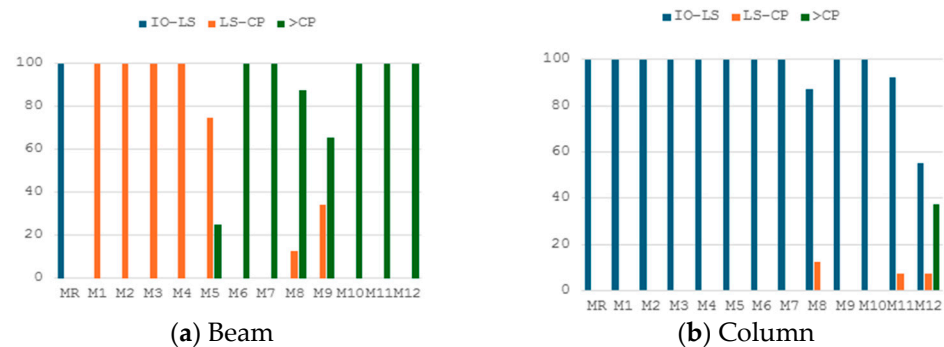


Figure 13. Damage levels for beam and columns.

4.5. Hinge Forming Mechanism

As a result of dynamic earthquake record analysis in SAP2000 [29], the plastic hinges that form in columns and beams indicate how much energy the structural elements can dissipate under a specific earthquake effect and how much deformation capacity they possess beyond elastic limits. These plastic hinges represent the points where the structure begins to yield under a certain load and exhibit a specific ductility capacity. Analyzing plastic hinges is important for evaluating the performance level of the structure. The locations where plastic hinges form reflect the ductility capacity of the elements and the target performance level of the building (e.g., ‘immediate occupancy’, ‘life safety’, ‘pre-collapse’). If hinges form at beam ends and are minimal or absent in columns, the structure is generally considered ‘safe’. However, if plastic hinges form in columns, the situation is more serious, as damage to columns can jeopardize the overall stability of the structure. Areas with a high concentration of plastic hinges show that the structure is dissipating energy in these regions, indicating that structural elements are yielding to absorb earthquake energy while maintaining the overall stability of the structure. The distribution of hinges throughout the structure is also significant. A uniform distribution suggests that the structure dissipates energy in a balanced manner, reducing the risk of sudden collapse. Conversely, the concentration of hinges in specific regions may indicate local damage and serious weaknesses. In this study, plastic hinges formed in beams and columns as a result of the applied earthquake load on all frame models were evaluated based on the resulting rotation values. Images showing the plastic hinge formation for

the reference model and the two models with the most adverse corrosion behavior are presented in Figures 14 and 15.

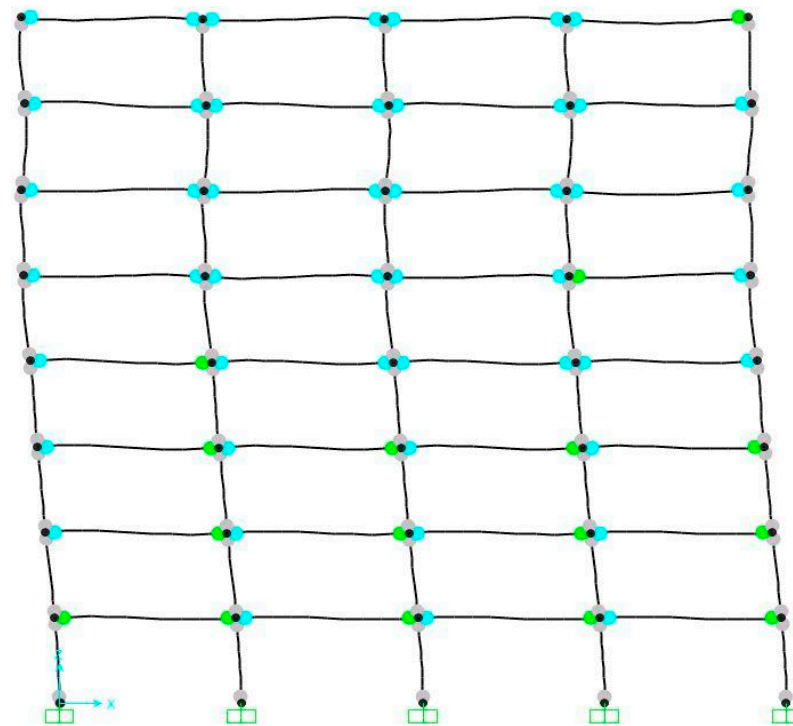


Figure 14. Hinge formation for the reference frame model.

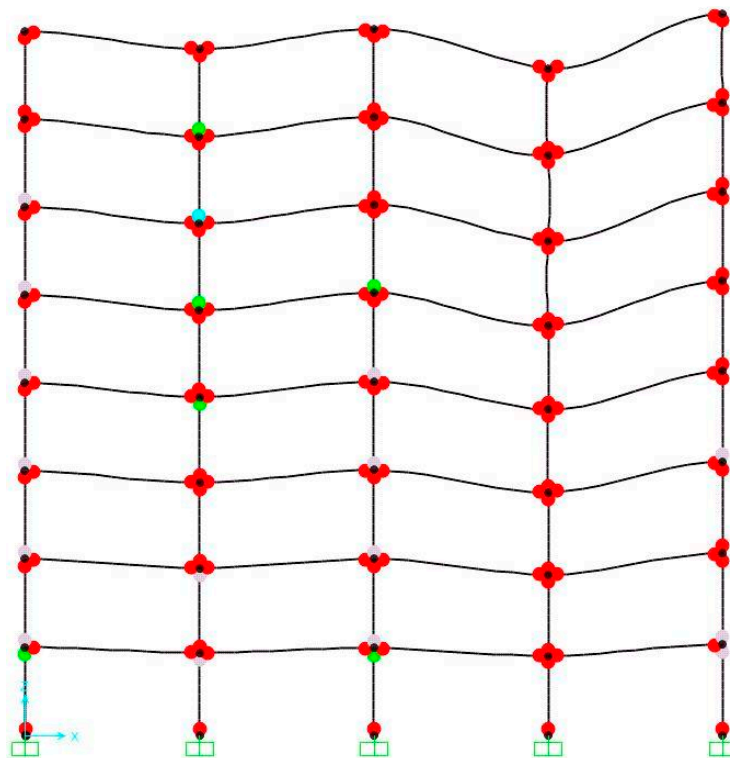


Figure 15. Hinge formation for the M12 frame model.

5. Conclusions

The corrosion initiation and propagation times of an RC frame were calculated using different methods. Based on the calculated times and selected corrosion current densities,

the structural performance of RC frames that reached their service life was examined. Additionally, the performance of RC frames with a 50-year service life under the same corrosion current densities was evaluated. The following conclusions were drawn from the analysis.

- Corrosion initiation times vary depending on the type of cement and the occurrence of cracks. Even the smallest crack of 0.1 mm in the frame shortened the corrosion initiation time by 58.3%.
- Corrosion propagation time depends on the water–cement ratio, cover thickness, reinforcement diameter, and corrosion current density. An increase in corrosion current density from $0.5 \mu\text{A}/\text{cm}^2$ to $6 \mu\text{A}/\text{cm}^2$ reduced the propagation time by 91%.
- Since the service life is the sum of the initiation and propagation times, this process is influenced by all the characteristics of the initiation–propagation process.
- In the case of a 0.1 mm crack in the frame, the service life of a frame with a corrosion current density of $0.5 \mu\text{A}/\text{cm}^2$ decreased by 20.6%, by 30.4% at $1 \mu\text{A}/\text{cm}^2$, by 46.7% at $3.5 \mu\text{A}/\text{cm}^2$, and by 50% at $6 \mu\text{A}/\text{cm}^2$. The service life of a section with a crack varies depending on the corrosion rate.
- Mass loss in the reinforcements due to corrosion increases with the corrosion rate and decreases with the reduction in reinforcement diameter. Using larger diameter reinforcements in areas exposed to chloride ions will reduce mass loss and, consequently, the detrimental effects of corrosion mechanisms.
- Corrosion reduces the base shear force but increases the top displacement and the displacement at the story levels. The main effect causing these changes is the corrosion rate. As the corrosion rate increases, the reinforcement diameter decreases, and the mechanical properties of the concrete and reinforcement deteriorate. This results in a decrease in the frame's load-carrying capacity.
- As corrosion effects increase, the structural period also increases. Since corrosion weakens the structure, the period duration increases. The effect of the corrosion rate on this increase is significant, while the effect of the propagation time is less.
- Moment and curvature decrease with corrosion. The increase in propagation time and corrosion rate leads to a loss of diameter in the reinforcement and deterioration in material properties, which increases damage in the section, thus reducing moment capacity.
- The increase in corrosion rate and propagation time causes an increase in the damage levels and hinges in beams and columns. Especially in regions with high seismic potential, where preventive measures against corrosion are essential.
- For an RC frame, with a service life of 50 years at a corrosion rate of 0.5, the service life is 34 years if there are no cracks and 27 years if cracks exist; at a corrosion rate of 1, the service life is 23 years if no cracks exist and 16 years if cracks are present; at a corrosion rate of 3.495, the service life is 15 years with no cracks and 8 years with cracks; and at 6 corrosion rate, the service life is 14 years with no cracks and 7 years with cracks. The structural performance in these years is the same. The presence of cracks and their thickness directly negatively impact the service life. The surfaces of RC elements should be monitored, and crack tracking should be carried out.
- The performance of RC structures exposed to aggressive environmental conditions for long periods will be adversely affected. Therefore, during the project phase, suitable materials should be selected and designed accordingly. The service life of RC buildings under chloride effects should be calculated based on environmental influences, and the years requiring maintenance should be determined and shared with the project owner.
- For structures exposed to aggressive environments, the surfaces of the elements should be protected against corrosion to prevent the ingress and accumulation of chlorides. This can be achieved by applying protective coatings, such as epoxy- or polymer-based treatments, which act as a barrier to harmful substances.
- In future studies, similar research can be carried out on shear walls rather than columns to investigate the differences between these components.

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