



Reliability-Based Modelling of Reinforced Concrete Structural Deterioration under Coupled Chloride-Induced Corrosion and Mechanical Loading

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ABSTRACT

Reinforced concrete (RC) structures exposed to chlorides while carrying sustained load deteriorate through interacting processes that are usually modelled separately. This study develops a time-dependent probabilistic framework in which chloride-induced corrosion and mechanical damage are coupled explicitly. A one-dimensional Fickian chloride ingress model with lognormal parameters defines the distribution of corrosion initiation time. After initiation, corrosion penetration is amplified by a mechanical damage index (one-way coupling), while the same damage index independently reduces flexural resistance. Resistance is compared with a time-growing bending-moment demand through the limit-state function $g(t) = M_R(t) - M_E(t)$, and Monte Carlo simulation (MCS) in MATLAB gives the probability of failure $P_f(t) = P[g(t) \leq 0]$ and the reliability index $\beta(t) = -\Phi^{-1}(P_f(t))$ over 50 years. For a simply supported 5.0 m beam, mean chloride concentration at the reinforcement rises to about 1.45 kg/m³ at 50 years and passes the 0.60 kg/m³ threshold at 19–20 years (5th–95th percentile at 50 years: 0.83–2.15 kg/m³). Bar diameter falls from 20.0 mm to 18.92 mm (–5.4%) for corrosion only and to 18.55 mm (–7.25%) for the coupled model, which is about 34% more diameter loss. Mean resistance falls from 67.5 to 39.5 kN·m (–41.5%) while mean demand rises from 80 to 89 kN·m (+11.3%), widening the resistance–demand gap from 12.5 to 49.5 kN·m. Because mean demand already exceeds mean resistance at $t = 0$, P_f starts near 0.85 ($\beta \approx -1.0$); under coupled deterioration β falls to about –3.4 and P_f approaches 1.0 by 30–35 years. The corrosion-attributable reliability difference reaches about 1.38 at 50 years. Neglecting corrosion–mechanical interaction therefore underestimates long-term deterioration in the case studied.

Keywords: reinforced concrete; chloride-induced corrosion; mechanical loading; coupled deterioration; structural reliability; Monte Carlo simulation

1. INTRODUCTION

Reinforced concrete is widely used for its strength, versatility and cost-effectiveness, yet it deteriorates under environmental and mechanical actions that reduce material quality, reinforcement capacity, serviceability and safety (Bastidas-Arteaga et al., 2018; Val et al., 2025). These processes are time-dependent and uncertain in materials, construction, exposure and loading, so reliable prediction is essential for infrastructure management (Verma et al., 2014; Zhang, 2018; Val et al., 2025).

Chloride-induced corrosion is a leading mechanism in marine settings and in structures exposed to de-icing salts or contaminated water (Angst et al., 2009). Once chloride at the reinforcement exceeds a critical content, the passive layer breaks down and corrosion begins, causing section loss, cracking, spalling and bond deterioration. Initiation is usually predicted with Fickian diffusion and a critical threshold, but deterministic models ignore the natural variability of diffusion coefficient, cover, surface chloride and corrosion rate (Saassouh & Lounis, 2012; Angst et al., 2009). Probabilistic models give a more realistic estimate of initiation, deterioration and failure likelihood (Val et al., 2025; Truong et al., 2022).

Time-dependent reliability studies show that uncertainty in corrosion initiation and progression can substantially reduce the reliability of RC beams (Li et al., 2024; Wu et al., 2024). Coupled models, such as the biodeterioration–chloride–cracking model of Bastidas-Arteaga et al. (2008), show that interacting mechanisms affect capacity significantly. However, most corrosion-reliability studies treat loading only as an applied action, and the explicit interaction between corrosion progression and mechanical damage remains under-explored.

This gap matters: treating the two mechanisms independently may underestimate deterioration and overestimate remaining reliability. The objectives of this study are therefore to: (i) formulate a stochastic chloride-ingress model for corrosion initiation; (ii) introduce an explicit, quantifiable coupling between mechanical damage and corrosion penetration; (iii) define the limit-state function, probability of failure and reliability index for the coupled system; and (iv) compare corrosion-only, mechanical-only and coupled predictions of reinforcement loss, resistance, P_f and β over 50 years.

2. MATERIALS AND METHOD

2.1 Overview and workflow

A simply supported RC beam represents the structural element. For each Monte Carlo sample and each year $t = 0, 1, \dots, 50$ the analysis proceeds as follows: (1) sample the random variables (Section 2.3); (2) compute chloride concentration at the bar and the initiation time T_i ; (3) compute corrosion penetration for the corrosion-only and coupled cases; (4) compute remaining bar area and flexural resistance; (5) compute demand; (6) evaluate $g(t)$; (7) accumulate $P_f(t)$ and $\beta(t)$. Three models are compared: **corrosion-only**, **mechanical-only** (no corrosion section loss) and **coupled**. The analysis was implemented in MATLAB.

2.2 Chloride transport model

Chloride ingress is represented by one-dimensional Fickian diffusion:

$$C(x,t) = C_s [1 - \operatorname{erf}(x / (2\sqrt{D_{\text{eff}} t}))] \quad (1)$$

where $C(x,t)$ is chloride concentration at depth x and time t (kg/m^3), C_s is surface chloride concentration (kg/m^3), D_{eff} is the effective diffusion coefficient (m^2/s), x is depth from the surface (m), t is exposure time (s) and erf is the error function. At the reinforcement, located at cover c (Saassouh & Lounis, 2012; Val et al., 2025):

$$C(c,t) = C_s [1 - \operatorname{erf}(c / (2\sqrt{D_{\text{eff}} t}))] \quad (2)$$

Assumptions. Diffusion is one-dimensional, D_{eff} and C_s are constant in time for a given sample (no ageing, temperature or binding effects), concrete is uncracked and saturated, and the surface concentration is reached instantaneously. These simplifications are common in first-order service-life models, and their consequences are discussed in Section 4.

2.3 Probabilistic parameters

The principal variables are random. For a positive variable X with mean μ_X and coefficient of variation δ_X a lognormal form is adopted, which excludes physically impossible negative values:

$$\sigma_{\ln X} = \sqrt{\ln(1 + \delta_X^2)} \quad (3)$$

$$\mu_{\ln X} = \ln \mu_X - \sigma_{\ln X}^2 / 2 \quad (4)$$

$$X = \exp(\mu_{\ln X} + \sigma_{\ln X} \cdot Z), \quad Z \sim N(0,1) \quad (5)$$

Table 2 lists the random variables. Variables not listed there are deterministic (Table 1).

Table 2. Random variables used in the Monte Carlo analysis

Variable	Symbol	Distribution	Mean	CoV
Surface chloride concentration	C_s	Lognormal	[insert]	[insert]
Effective diffusion coefficient	D_{eff}	Lognormal	[insert]	[insert]
Concrete cover	c	Lognormal	0.050 m	[insert]
Critical chloride concentration	C_{cr}	[insert]	0.60 kg/m ³	[insert]
Corrosion penetration rate	r_{corr}	Lognormal	[insert]	[insert]
Live-load growth coefficient	g_L	[insert]	[insert]	[insert]
Resistance model uncertainty	θ_R	Lognormal	[insert]	[insert]

2.4 Corrosion initiation

Corrosion starts when chloride at the bar reaches the critical concentration:

$$C(c,t) \geq C_{cr} \quad (6)$$

Inverting Eq. (2), the initiation time of each sample is

$$T_i = c^2 / (4 D_{eff} [\text{erf}^{-1}(1 - C_{cr}/C_s)]^2) \quad (7)$$

Because the inputs are random, T_i is a distribution, not a single value.

2.5 Corrosion propagation (corrosion-only model)

After initiation, radial corrosion penetration grows at an effective rate r_{corr} :

$$\delta_{corr}(t) = 0 \text{ for } t \leq T_i \quad (8)$$

$$\delta_{corr}(t) = r_{corr} (t - T_i) \text{ for } t > T_i \quad (9)$$

The remaining bar diameter, with physical lower bound, and area are

$$d_r(t) = d_0 - 2\delta(t) \quad (10)$$

$$d_r(t) = \max(d_r(t), 0) \quad (11)$$

$$A_s(t) = \pi d_r(t)^2 / 4 \quad (12)$$

where $d_0 = 20$ mm is the initial bar diameter.

2.6 Mechanical deterioration

Sustained and repeated loading is represented by a bounded, phenomenological damage index:

$$D_m(t) = 1 - \exp(-\lambda t) \quad (13)$$

where $D_m \in [0,1)$ and λ is the mechanical deterioration rate (1/year). The index is not derived from a fatigue or cracking law; it is a calibrated surrogate for cumulative load-induced damage (e.g. microcracking, bond and stiffness loss).



2.7 Coupled corrosion–mechanical model

The novelty of the study is an explicit link between mechanical damage and corrosion penetration. In the coupled case the penetration is amplified by the damage state:

$$\delta_c(t) = \Gamma(t) \cdot r_{corr}(t - T_i), \quad t > T_i \quad (14)$$

$$\Gamma(t) = 1 + \kappa D_m(t) \quad (15)$$

$$\delta_c(t) = r_{corr}(t - T_i)[1 + \kappa D_m(t)] \quad (16)$$

where κ is the corrosion–mechanical coupling coefficient and Γ is the amplification factor. Physically, mechanical damage (cracking, loss of cover integrity, bond degradation) provides easier access for chlorides, oxygen and moisture, so corrosion advances faster in damaged concrete (Bastidas-Arteaga et al., 2008). For $\kappa = 0$ the model reduces to the corrosion-only case.

The coupling-induced extra penetration and the relative extra diameter loss are

$$\Delta\delta(t) = \delta_c(t) - \delta_{corr}(t) \quad (17)$$

$$\Delta\delta(t) = \kappa r_{corr}(t - T_i) D_m(t) \quad (18)$$

$$\Delta d_{rel}(t) = 2\Delta\delta(t) / d_o \quad (19)$$

Eq. (19) isolates the reinforcement deterioration attributable to the interaction.

2.8 Flexural resistance

For the simply supported beam, effective depth and equivalent rectangular stress-block depth are

$$d = h - c - d_o/2 \quad (20)$$

$$a = A_s f_y / (0.85 f_c' b) \quad (21)$$

where h is overall depth, f_y is steel yield strength, f_c' is concrete compressive strength and b is beam width. Nominal flexural resistance is

$$M_R = A_s f_y (d - a/2) \quad (22)$$

For the corrosion-only case, A_s is replaced by $A_{s,corr}(t)$ from Eqs. (10)–(12):

$$M_{R,corr}(t) = A_{s,corr} f_y (d - a_{corr}/2) \quad (23)$$

2.9 Coupled resistance

Mechanical damage reduces resistance independently of section loss through a reduction factor:

$$\phi_m(t) = 1 - \eta D_m(t) \quad (24)$$

where η is the sensitivity of resistance to mechanical damage. Coupled resistance, using the amplified-corrosion area $A_{s,c}(t)$ from δ_c , is

$$M_{R,c}(t) = \theta_R \phi_m(t) A_{s,c} f_y (d - a_c/2) \quad (25)$$

where θ_R is model and resistance uncertainty.



2.10 Time-dependent demand

The bending moment grows slowly with service time because of changing use and accumulated loading (Li et al., 2024):

$$M_E(t) = M_D + M_{L0} (1 + g_L t) \quad (26)$$

where M_D is the dead-load moment, M_{L0} the initial live-load moment and g_L the stochastic live-load growth coefficient. Mean demand rises from 80 to 89 kN·m over 50 years.

2.11 Limit-state function

Safety is defined by comparing resistance and demand:

$$g(t) = M_R(t) - M_E(t) \quad (27)$$

with $g > 0$ the safe state, $g = 0$ the limit state and $g \leq 0$ failure. For the models compared:

$$g_{\text{corr}}(t) = M_{R,\text{corr}}(t) - M_E(t) \quad (28)$$

$$g_c(t) = M_{R,c}(t) - M_E(t) \quad (29)$$

Eq. (29) is the principal limit state of the coupled analysis. Resistance decreases with t through the A_s and ϕ_m terms of Eq. (25), while demand increases through Eq. (26); the safety margin $E[g] = E[M_R] - E[M_E]$ therefore shrinks from both sides.

2.12 Probability of failure

For N Monte Carlo samples:

$$P_f(t) = (1/N) \sum_i I[g_i(t) \leq 0] \quad (30)$$

$$I[\cdot] = 1 \text{ if } g_i(t) \leq 0, \text{ and } 0 \text{ otherwise} \quad (31)$$

$$P_{f,c}(t) = P[g_c(t) \leq 0] \quad (32)$$

2.13 Reliability index and its interpretation

$$\beta(t) = -\Phi^{-1}(P_f(t)) \quad (33)$$

$$\beta_c(t) = -\Phi^{-1}(P_{f,c}(t)) \quad (34)$$

where Φ^{-1} is the inverse standard normal CDF. **Interpretation:** $\beta > 0$ means $P_f < 0.5$ (mean resistance exceeds mean demand); $\beta = 0$ corresponds to $P_f = 0.5$; **$\beta < 0$ means $P_f > 0.5$, i.e. the section is more likely to be overloaded than not.** For example, $P_f = 0.85$ gives $\beta \approx -1.04$, and $P_f = 0.98$ gives $\beta \approx -2.05$. A negative β is therefore mathematically valid and signals that the safety margin is already exhausted; it is not a numerical error. Since $\beta \rightarrow -\infty$ as $P_f \rightarrow 1$, values near -3.4 require N large enough to resolve $P_f \approx 0.9997$ ($N \geq [\text{insert}]$).

2.14 Reliability-difference indicators

To separate the contribution of each mechanism, two indicators are used (Fig. 8):

$$\Delta\beta_{\text{corr}}(t) = \beta_{\text{corr}}(t) - \beta_c(t) \quad (35)$$

$$\Delta\beta_{\text{mech}}(t) = \beta_{\text{mech}}(t) - \beta_c(t) \quad (36)$$



where β_{corr} and β_{mech} are the corrosion-only and mechanical-only indices. Positive $\Delta\beta$ means the coupled model is less reliable than the comparison model.

2.15 Sensitivity analysis

A one-at-a-time sensitivity analysis is performed about the baseline case: each of C_s , D_{eff} , c , C_{cr} , r_{corr} , λ , κ , η , f_y , f_c' and g_L is varied by $\pm 20\%$ of its mean while the others are fixed, and the change in β at 25 and 50 years ($\Delta\beta$) is recorded and ranked in a tornado chart (Fig. 9, Table 5).

2.16 Model assumptions and limitations

1. Chloride transport is one-dimensional Fickian diffusion with time-invariant D_{eff} and C_s per sample.
2. C_s and D_{eff} are lognormal random variables; corrosion starts when $C(c,t)$ exceeds C_{cr} .
3. Mechanical damage follows the bounded exponential law of Eq. (13).
4. The coupling is **one-way**: damage amplifies corrosion (Eq. 15) and independently reduces resistance (Eq. 24); corrosion does not feed back into D_m .
5. Flexural resistance uses an equivalent rectangular stress block; shear, bond and cracking-induced serviceability limit states are not checked.
6. Corrosion is assumed uniform along the bar (no pitting, no spatial variability).
7. Failure is the flexural limit state $g \leq 0$; uncertainties in material properties, deterioration parameters, loading and model resistance are propagated by MCS.

3. RESULTS AND DISCUSSION

Table 1. Deterministic simulation parameters

Parameter	Value
Beam width, b	0.30 m
Beam overall depth, h	0.50 m
Beam span, L	5.0 m
Concrete cover, c	0.050 m
Surface chloride concentration, C_s	0.20 kg/m ³ [verify, see note]
Concrete compressive strength, f_c'	30 MPa
Concrete elastic modulus	30 GPa
Steel yield strength, f_y	500 MPa
Steel elastic modulus	200 GPa
Initial bar diameter, d_o	20 mm
Critical chloride concentration, C_{cr}	0.60 kg/m ³
Service life	50 years

3.1 Chloride ingress and corrosion initiation (Fig. 1)

Mean chloride concentration at the reinforcement rises from near zero to about 1.45 kg/m³ at 50 years and exceeds the 0.60 kg/m³ threshold at about 19–20 years. The 95th percentile reaches 1.13 kg/m³ at 20 years and 2.15 kg/m³ at 50 years, crossing the threshold at about 12 years. The 5th percentile stays below the threshold until about 36–37 years and reaches 0.83 kg/m³ at 50 years. Threshold crossing therefore spans roughly 12–37 years, against about 20 years for the mean response, which confirms that a single deterministic initiation time hides a wide range of outcomes.

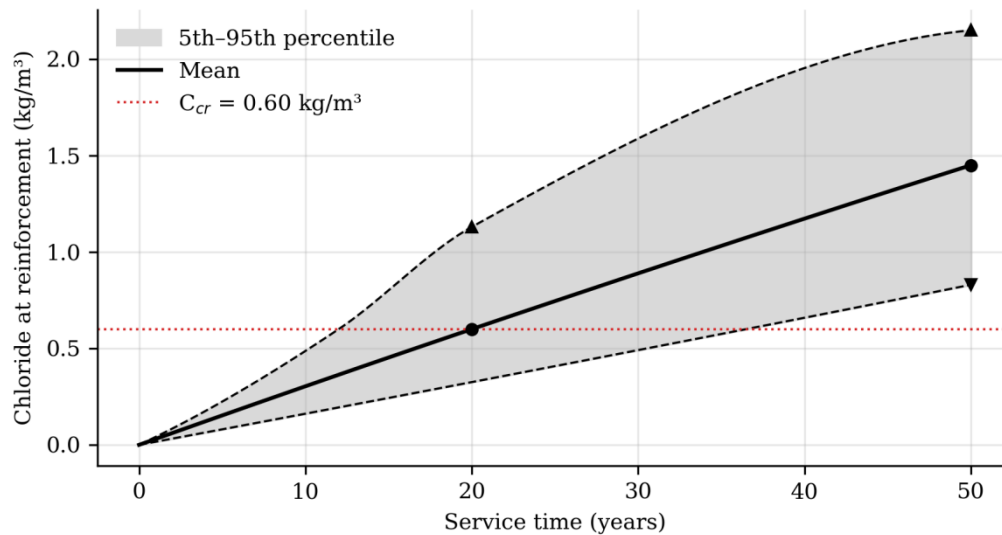


Fig. 1. Mean and 5th–95th percentile chloride concentration at reinforcement depth over 50 years, with the critical threshold. Markers: values reported from the Monte Carlo analysis; lines: monotone interpolation between them.

Table 3. Chloride concentration at reinforcement depth (kg/m³)

Statistic	At 20 years	At 50 years	Threshold crossing (years)
Mean	0.6	1.45	19–20
95th percentile	1.13	2.15	≈12
5th percentile	–	0.83	36–37

3.2 Reinforcement deterioration (Fig. 2)

Both models start at 20.0 mm. At 30 years the diameters are 19.70 mm (corrosion-only) and 19.60 mm (coupled); at 40 years, 19.30 mm and 19.10 mm. At 50 years the corrosion-only model gives 18.92 mm (loss 1.08 mm, 5.4%) and the coupled model 18.55 mm (loss 1.45 mm, 7.25%). The coupling thus adds 0.37 mm of loss, about 34% more than corrosion alone, and the gap widens with time as D_m approaches its bound and Γ increases (Eq. 15).

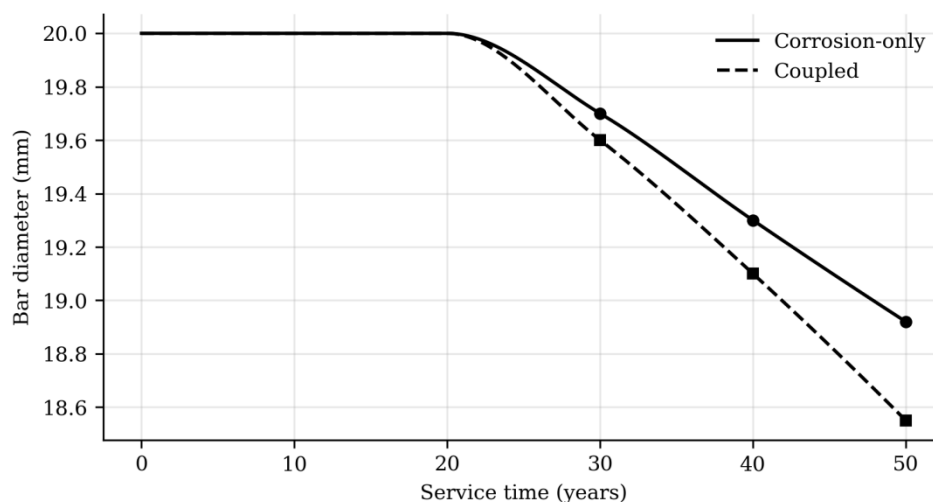


Fig. 2. Reinforcement diameter versus service time for the corrosion-only and coupled models. Markers: values reported from the Monte Carlo analysis; lines: monotone interpolation between them.

3.3 Structural resistance (Fig. 3)

All models start at about 67.5 kN·m. At 50 years the corrosion-only model has fallen to about 60.0 kN·m (−11.1%), the mechanical-only model to about 41.2 kN·m (−39.0%) and the coupled model to about 39.5 kN·m (−41.5%). The coupled curve converges with the mechanical-only curve around 35–37 years and declines faster afterwards, ending about 1.7 kN·m below it. Resistance loss is thus dominated by the mechanical term ϕ_m , while corrosion adds a smaller but growing section-loss contribution once the bar area has reduced appreciably.

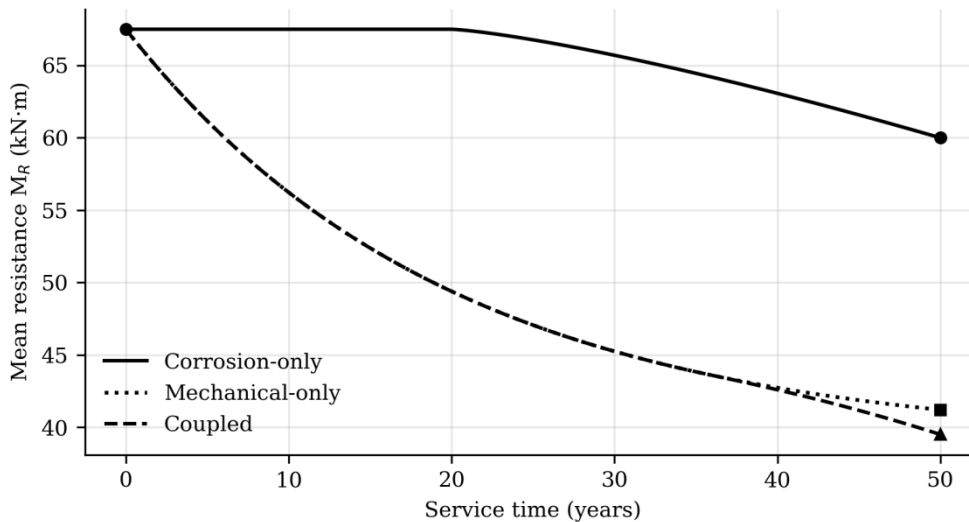


Fig. 3. Mean flexural resistance versus service time for the corrosion-only, mechanical-only and coupled models. Markers: values reported from the Monte Carlo analysis; lines: monotone interpolation between them.

3.4 Resistance versus demand (Fig. 4)

Mean resistance falls from 67.5 to 39.5 kN·m (−28.0 kN·m, −41.5%) while mean demand rises from 80 to 89 kN·m (+9 kN·m, +11.3%). The resistance–demand gap therefore widens from 12.5 kN·m at $t = 0$ to 49.5 kN·m at 50 years. Of this 37 kN·m growth in gap, about 76% (28/37) comes from resistance loss and 24% (9/37) from demand growth, so deterioration of the section, not load growth, governs the loss of safety margin. Note that mean demand already exceeds mean resistance at $t = 0$, which is why P_f and β in Section 3.5 start at 0.85 and −1.0.

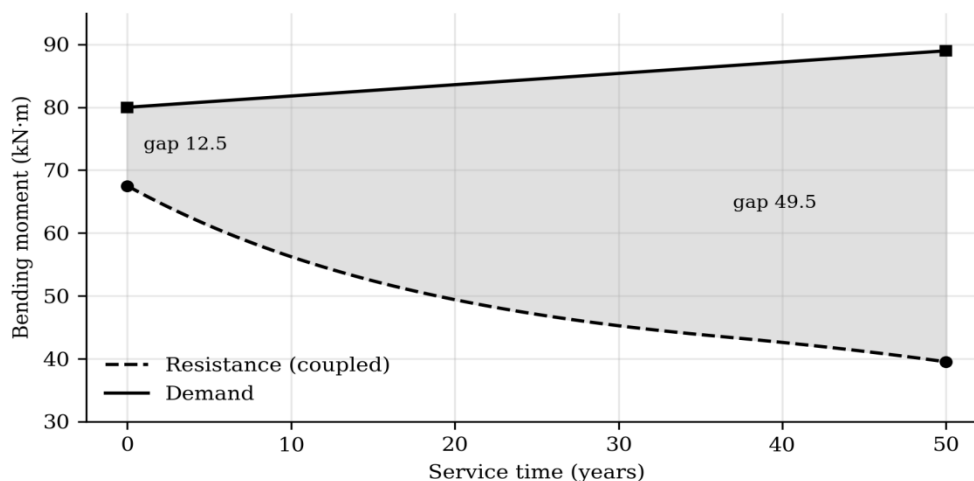


Fig. 4. Coupled mean resistance versus mean demand, showing the widening resistance–demand gap. Markers: values reported from the Monte Carlo analysis; lines: monotone interpolation between them.

3.5 Probability of failure and reliability index (Figs. 5–7)

Failure probability increases with age for all models (Fig. 5).

- *Mechanical-only*: about 0.85 at 0 years, 0.94 at 7 years, 0.97 at 15 years, above 0.98 by 20 years, approaching 1.00 by 30–35 years.
- *Coupled*: about 0.92 at 7 years, 0.95 at 10 years, 0.97 at 15 years and 0.98–0.985 at 20 years, approaching 1.00 by 30–35 years.
- *Corrosion-only*: slower, from 0.85 to 0.88 at 15 years, 0.92 at 25 years, 0.94 at 32 years and 0.98 at 50 years.

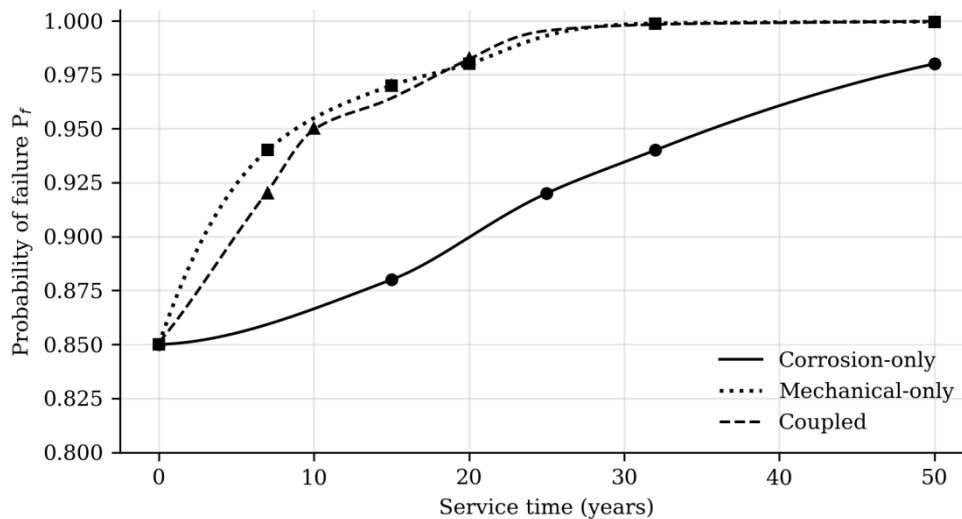


Fig. 5. Probability of failure versus service time for the three models. Markers: values reported from the Monte Carlo analysis; lines: monotone interpolation between them.

In terms of β (Fig. 6), all models start at about -1.0 . The corrosion-only model declines to about -2.0 at 50 years and the mechanical-only model to about -3.3 . The coupled model reaches about -1.8 at 15 years, -2.6 at 25 years, -3.0 at 35 years and -3.4 to -3.5 at 50 years. Fig. 7 shows the coupled β falling from -1.0 to -3.4 (-2.4 units), passing through about -2.1 at 20 years and -3.1 at 40 years, and remaining far below the reference levels $\beta = 2.0$, 2.5 and 3.0 throughout. The coupled curve is below the corrosion-only curve at all ages and, at 50 years, slightly below the mechanical-only curve.

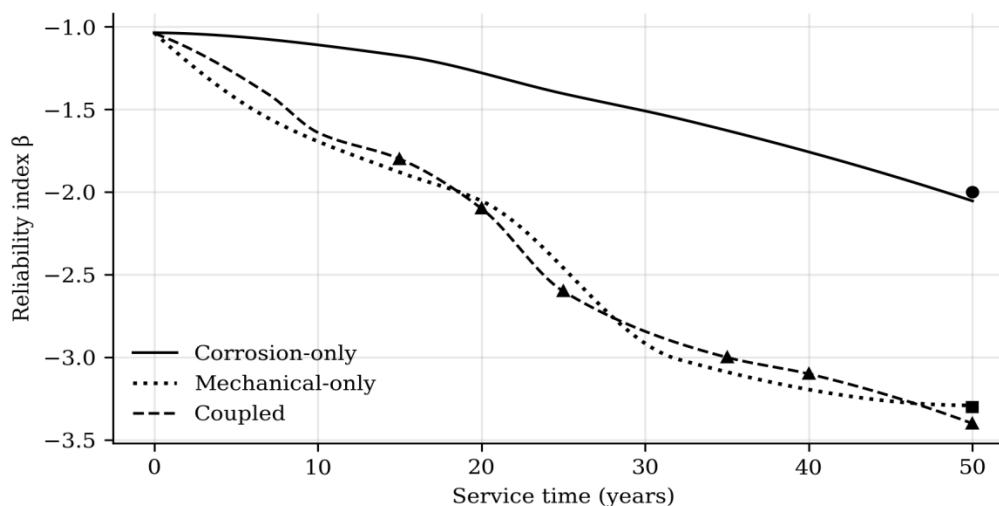


Fig. 6. Reliability index versus service time for the three models. Markers: values reported from the Monte Carlo analysis; lines: monotone interpolation between them.

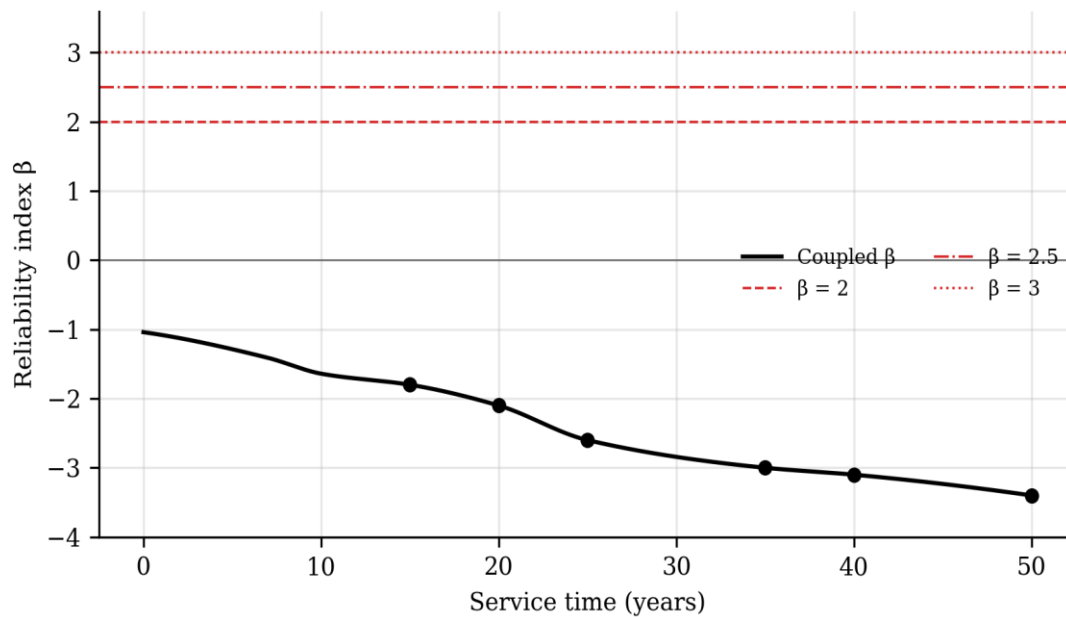


Fig. 7. Coupled reliability index against reference levels $\beta = 2.0, 2.5$ and 3.0 . Markers: values reported from the Monte Carlo analysis; lines: monotone interpolation between them.

The negative values follow from Eqs. (27)–(33): when mean demand exceeds mean resistance, $P_f > 0.5$ and $\beta < 0$ (Section 2.13). They indicate that the example section is overloaded from the outset and degrades further, not that the index is mis-computed. Absolute β should therefore not be compared with code target values (typically β of about 3–4 for 50-year reference periods); the useful result is the *relative* acceleration of reliability loss between models.

Table 4. Coupled-model reliability index and failure probability at selected ages

Service time (years)	β (reported)	$P_f = \Phi(-\beta)$
0	≈ -1.0	≈ 0.85
15	-1.8	0.9641
20	-2.1	0.9821
25	-2.6	0.9953
35	-3	0.9987
40	-3.1	0.999
50	-3.4	0.9997

P_f at $t = 0$ as reported; other values computed from the reported β using Eq. (33).

3.6 Contribution of each mechanism (Fig. 8)

$\Delta\beta_{\text{corr}}$ rises from 0.0 at year 0 to about 1.0 at 25 years, 1.2 at 35 years and 1.38 at 50 years, so the coupled model increasingly departs from the corrosion-only prediction. $\Delta\beta_{\text{mech}}$ reaches a minimum of about -0.31 between 20 and 23 years and recovers to about 0.12 at 50 years. The negative interval means that, at these ages, the mechanical-only model gives a lower β than the coupled model. This is attributable to sampling and the saturation of P_f near 1.0, where β is highly sensitive to small differences in P_f ; it also shows that at mid-life the mechanical term dominates and the coupling adds little. The corrosion effect, in contrast, grows monotonically and is the main contributor to the coupled–separate difference at long service times.

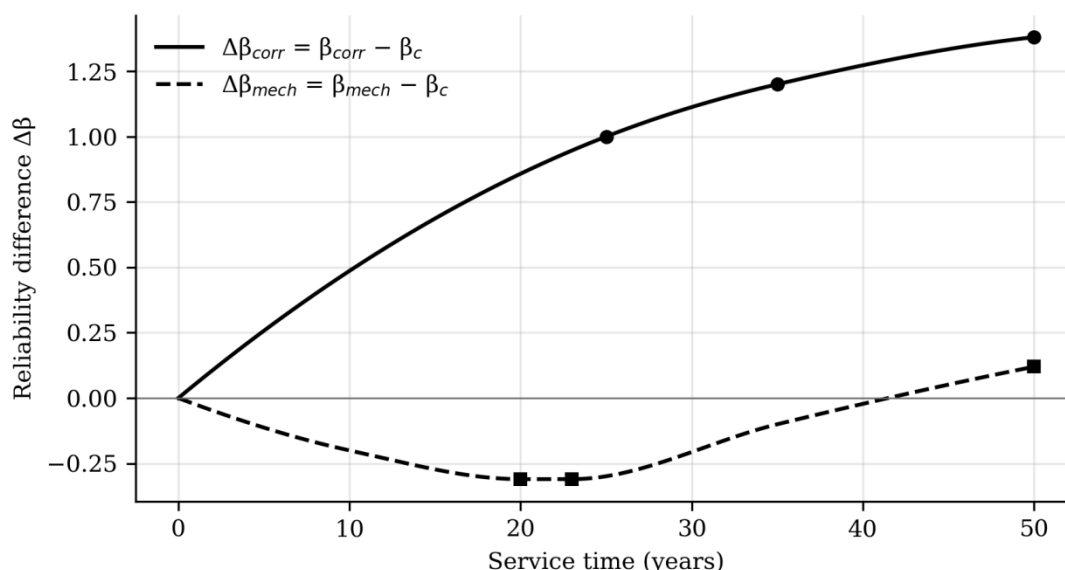


Fig. 8. Reliability differences attributable to corrosion ($\Delta\beta_{corr}$) and mechanical loading ($\Delta\beta_{mech}$). Markers: values reported from the Monte Carlo analysis; lines: monotone interpolation between them.

3.7 Sensitivity analysis (Fig. 9, Table 5)

Table 5. Change in β (coupled model) for $\pm 20\%$ variation in each parameter

Parameter	$\Delta\beta$ at 25 years	$\Delta\beta$ at 50 years	Rank
[to be completed from MCS runs]			

The tornado chart will identify which of the inputs in Section 2.15 govern long-term reliability, and therefore which should be measured or controlled in practice.

3.8 Summary of principal results

Table 6. Comparison of models at 50 years

Quantity	Corrosion-only	Mechanical-only	Coupled
Bar diameter (mm)	18.92	—	18.55
Diameter loss	5.4%	—	7.25%
Mean resistance (kN·m)	≈ 60.0	≈ 41.2	≈ 39.5
Resistance reduction	11.1%	39.0%	41.5%
Mean demand (kN·m)	89	89	89
Failure probability	≈ 0.98	≈ 1.00	≈ 1.00
Reliability index β	≈ -2.0	≈ -3.3	≈ -3.4

4. LIMITATIONS

The results come from one illustrative beam. The coupling coefficient κ , the damage-rate parameter λ and the sensitivity η are phenomenological and were not calibrated to test data, so the magnitude of the coupling effect (+34% diameter loss) should be read as model-dependent. Uniform corrosion, one-way coupling, time-invariant diffusion and the flexural-only limit state are further simplifications. Since the initial demand exceeds resistance, the case represents a heavily loaded or under-designed member, and a design-compliant section (β_0 near the code target) should be analysed in future work. Field or laboratory calibration and spatially variable corrosion are recommended next steps.



5. CONCLUSION

Combined chloride-induced corrosion and mechanical loading strongly affect the long-term reliability of RC beams. Stochastic chloride ingress shows wide uncertainty in corrosion initiation (about 12–37 years). Coupling mechanical damage with corrosion increases 50-year diameter loss from 5.4% to 7.25% and reduces mean resistance by 41.5%, while demand growth of 11.3% enlarges the resistance–demand gap to 49.5 kN·m. Reliability falls continuously (β from about -1.0 to -3.4) with P_f approaching 1.0 by 30–35 years, and the corrosion-attributable reliability difference reaches about 1.38 at 50 years. Explicit limit-state, probability and reliability-index definitions, together with the sensitivity analysis, make the framework a basis for condition monitoring, risk-informed maintenance planning and service-life assessment. Neglecting the interaction between mechanisms can underestimate long-term vulnerability.

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