

TIME VARIANT RELIABILITY ANALYSIS OF REINFORCED CONCRETE BRIDGES SUBJECTED TO BOTH LOADING AND CHLORIDE INDUCED DEGRADATION

L. D. Nyihemba¹, I. Abubakar², S. P. Ejeh³ A. Ocholi⁴

¹ Federal Roads Maintenance Agency (FERMA), Katsina state, Nigeria, +234-8036910018 (corresponding author) nyihemba@yahoo.com

^{2, 3, 4}Department of Civil Engineering, Ahmadu Bello University, Zaria, Nigeria.

Abstract

The reliability of the structural performance of reinforced concrete bridges is affected by aging, aggressive environments and traffic loadings. Time-variant reliability analysis of a two span reinforced concrete bridge was investigated using First Order Reliability Method (FORM). Limit state functions considering ten failure modes for members in bending were considered to accommodate the time-dependent effects in the structural reliability analysis. The entire process was implemented via a developed program using MATLAB. The program is automated to calculate the reliability indices using the limit functions for each of the ten failure modes over the bridge service life of 120years at 10years intervals. The random variables are imputed directly through the interactive part of the program while other functions are imbedded in the main program input directory. The results show that the load capacity loss ranges between 9.41% for the least deteriorated member failure mode to 100% for the most deteriorated member failure mode as the corrosion rate increases from 0.02 to 0.06 mm/year at load growth rate of 0.005. Lower capacity losses were however obtained at a constant load (no load growth), with losses ranging from 0.25% for the least degraded member failure mode to 34.47% for the most deteriorated member failure mode as the corrosion rate increases from 0.02 to 0.06 mm/year. The load capacity of the most degraded member, Failure mode 1: Failure of the deck in bending and other degrading members can be enhanced if preventive measures are put in place to stem the progressive effect of corrosion over the service life of the structure.

Keywords: Structural performance, Corrosion, Load growth, Capacity loss, Reinforced Concrete Bridge.

1. Introduction.



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Time changes the resistance of a bridge due to environmental factors, but many reliability studies on reinforced concrete bridges do not factor in “Time –dependence” aspects. It is strongly affected during service by many deteriorating factors. The live load – truck weights and numbers per day is expected to increase over time (Bigaud *et al.*, 2014), and the bridge deteriorates through aging, increased use, and specific mechanisms such as fatigue and corrosion (Estes and Frangopol, 1999). Studies have found that corrosion causes deterioration in the properties of steel – area, yield strength and ultimate strain (Almusallam, 2001; Cairns *et al.*, 2005), loss of bonding between concrete and steel bars, cracking and spalling of the concrete cover (Liu and Weyers, 1998; Li, 2006).

A bridge structure is a complex system composed of many inter-related bridge elements (Deck, super structure and sub structure). Each element contributes to the overall performance or safety of the system. Both the resistance and loading effects of a bridge structure are time-dependent variables and must be considered in the service-life prediction of the deteriorating structures (Bordallo-Ruiz, 2007). The deterioration of the mechanical properties of structural systems under environmental attacks may be dealt with as a reliability problem where every loss of performance greater than prescribed threshold values is considered as a “failure” (Sarja,1996).

Also there exist uncertainties in material and geometrical properties, in the physical models of deterioration process, and in mechanical and environmental stressors, a measure of the time-variant structural performance is realistically possible only in probabilistic terms (Ang, and Tang, 2007; Frangopol and Ellingwood 2010; Biondini and Frangopol, 2014). Majority of studies have focused on time-variant reliability of reinforced concrete bridge elements subjected to corrosion induced deterioration (Tarighat. and Jalalifar, 2013) and combine effect of corrosion and load (Bigaud *et al.*, 2014; Bordallo-Ruiz *et al.*, 2007) on the structural reliability over time. These studies utilize the critical load effect rather than progressive load increment (load growth) during the service life of the structure. This paper discusses the time-variant reliability analysis of reinforced bridges subjected to both progressive load increment and chloride induced degradation throughout the service life of the structure. It also assesses the performance of the bridge elements in bending over its service life with a view to identifying the critical elements which affect the safety of the entire bridge structure.



2. Time – Variant Resistance.

In general, time – dependent resistance of an element can be expressed as a product of the initial resistance and a resistance degradation function (Mori and Ellingwood 1993).

$$R(t) = R_o g(t) \quad (1)$$

Where in equation (1); R_o is the initial resistance and $g(t)$ is the resistance degradation function. Degradation of resistance is brought about by a reduction in reinforcement area and changes in steel mechanical properties originated by corrosion.

2.1 Area Loss of Steel Reinforcement and Degradation of Material Properties Due to Corrosion.

Mathematical model of structural capacity loss as a function of the loss of reinforcing steel cross – sectional area in reinforced concrete structures is well documented in literature (Cady and Weyers, 1984; DNV-OS-C50 (2010); Frangopol and Moses 1994; Frangopol and Hendawi 1994; Thoft-Christensen, 2000; Li, 2005; Vu and Stewart, 2005; Kupwade-Patil *et al.*, 2012; Adamu, *et al.*, 2014). For a reinforced concrete element with equal diameter bars, subject to the same corrosion initiation times, the time – variant area of steel reinforcement can be expressed as given in Equations 2-4 (Bordallo-Ruiz *et al.*, 2007):

$$A(t) = \frac{n\pi D_i^2}{4} \quad (\text{for } t \leq T_i) \quad (2)$$

$$A(t) = \frac{n\pi D(t)^2}{4} \quad (\text{for } T_i \leq t \leq T_i + \frac{D_i}{r_{\text{corr}}}) \quad (3)$$

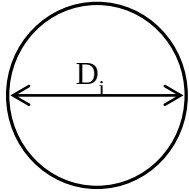
$$A(t) = 0 \quad (\text{for } t \geq T_i + \frac{D_i}{r_{\text{corr}}}) \quad (4)$$

Where in Equation (2) to (4) n is the number of reinforcing bars, D_i is the initial diameter of steel reinforcement shown in Figure 1(a), t is the elapsed time, r_{corr} is the corrosion rate (mm/year), T_i is the corrosion initiation time.

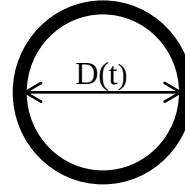


$$D(t) = D_i - 2r_{\text{corr}}(t - T_i) \quad (5)$$

$D(t)$ is the diameter of a bar under corrosion shown in Figure 1(b).



(a) Before corrosion



(b) During corrosion

Figure 1: Section Showing the Diameter of Reinforcement Bar.

The factor 2 in Equation (5) takes into account the uniform corrosion propagation process from all sides at the level of rebar.

It is worth mentioning that corrosion does not affect the steel area only, but its actions also change steel mechanical properties (Bigaud *et al.*, 2014). The yield strength at age t has been assumed by Cairns *et al.* (2005) to be linearly proportional to the reduced cross-sectional area $A(t)$ such that:

$$f_y(t) = \left(1 - \alpha_y \frac{A_s(t)}{A_{s0}}\right) f_{y0} \quad (6)$$

Where, f_{y0} is the initial steel yield stress, A_{s0} is the initial bar area and α_y is an empirical factor. 'A review of twelve experimental studies in [3] reports an average value of empirical factors up to 0.01, which has been adopted in this study.

2.2 Time dependent Live Load Model.

The Virginia Department of Transportation, following the results of numerous surveys strongly recommends the compound growth function for truck loads [1].

$$TGVW(t) = TGVW_{\text{initial}} * (1 + \lambda_w)^t \quad (7)$$

Where $TGVW(t)$ is the truck gross vehicle weight at age t (in years), λ_w is the weight growth rate in percentage, $TGVW_{\text{initial}}$ is the truck gross vehicle weight at the construction time of the bridge.



The time-dependent mean truck weight is given by Vu (2000):

$$\mu_w(t) = \mu_w * (1 + \lambda_m)^t \quad (8)$$

Where $\mu_w(t)$ is the truck weight at age t (in years), λ_m is the annual increase in truck weight (= 0.005), that gives a 65% increase in weight after 100 years and μ_w is the initial truck weight at the construction time of the bridge, t is the time in years.

2.4 Probability of Failure and Reliability Index

During service life, the reliability of bridge structures decrease due to the degradation of resistance and the increment in the designed traffic loads over time. The cumulative probability of failure and reliability index over the bridge's service life is usually calculated by:

$$P_f(t) = P[g(R(t), Q(t)) < 0] = P\left[\frac{R(t)}{Q(t)} < 1\right] \quad (9)$$

And,

$$\beta(t) = \Phi^{-1}[1 - P_f(t)] \quad (10)$$

Where Φ^{-1} is the inverse standard normal distribution function, $P_f(t)$ is the probability of failure at time t , and $\beta(t)$ is the reliability index at time t .

1. Generally, if $R(t)$ (Resistance or capacity) at time t and $Q(t)$ (Demand) at time t are uncorrelated random variables, the reliability index can be calculated (Hasofer, 1974).

$$\beta(t) = \frac{\mu_{R(t)} - \mu_{Q(t)}}{\sqrt{(\sigma_{R(t)}^2 + \sigma_{Q(t)}^2)}} \quad (11)$$

Where μ_R and μ_S are the mean values of $R(t)$ and $Q(t)$, and $\sigma_{R(t)}$ and $\sigma_{Q(t)}$ are the standard deviations of $R(t)$ and $Q(t)$, respectively.

3. Materials and Method

Basic principles of engineering structural mechanics were applied using the provisions of the relevant Eurocodes (EN 1992-1-1. (2004) and EN 1992-2.(2005) for materials. EN 1991-1-1. (2002) ; EN 1991-1-5.(2004); EN 1991-2.(2003); EN 1997-1.(2004) for actions on the structure,



while load combinations are derived from EN 1990 (2002) [31]). A MATLAB based program was developed and used in carrying out the components reliability analysis. The following assumptions have been made in this study:

- All sides of the structure are exposed to an aggressive environment and subjected to the same degree of corrosion.
- A deteriorating process with no damage of concrete and uniform corrosion of steel bars is considered (Biondini and Frangopol, 2013).
- The corrosion rate of steel depends on the concentration of the aggressive agent (Bertolini *et al.*, 2004). This varies from environment to environment.
- Resistance loss due to concrete cracking and spalling is ignored.
- A constant corrosion rate is assumed over the service life of the structure.
- In classical structural analysis models, perfect bond strength between steel and concrete was assumed, thus for coherence and simplicity, bond strength loss is not considered. In practice, the corrosion of steel bars is a combination of general and pit (localised) corrosion. And bond strength loss could more or less affect the resistance capacity of a structure (Val *et al.*, 1998)

3.1 Bridge Model

A simply supported bridge whose cross-section is shown in Figure 2. It consists of two equal spans of 15.0 m each which covers an effective length of 30.0 m is located in an open area and is characterized by an open cross section composed of seven (7) precast reinforced concrete longitudinal beams set at constant spacing of 1.70 m.

The upper flanges of the precast longitudinal beams are duly connected to a 0.18 m deep in-situ deck slab cast on a 0.07 m thick precast concrete slab formwork, giving a deck slab thickness of 0.25m. The superstructure is integrated with the substructure via bearing pads. The foundation for the bridge consists of cast in-situ reinforced concrete piles with pile caps. The



total width of the bridge is 11.0m. The carriage way is 7.30 m wide, and has a walkway on each side of 1.5 m wide.

Materials are chosen according to (EN 1992-1-1, 2004; EN 1992-2, 2005). The strength class of structural normal weight concrete f_{ck} is C25/30. The reinforcing steel strength f_{yk} is B500C. Density of reinforced Concrete $\gamma_{conc} = 25.0 \text{ kN/m}^3$, Density of asphalt concrete $\gamma_{asph} = 23 \text{ kN/m}^3$, Weight of parapet wall = 0.5 kN/m . Actions on the bridge are determined according to (EN 1991-1-1, 2002; EN 1991-1-5, 2004; EN 1991-2, 2003; EN 1997-1, 2004) and load combinations derived from (EN 1990, 2002).

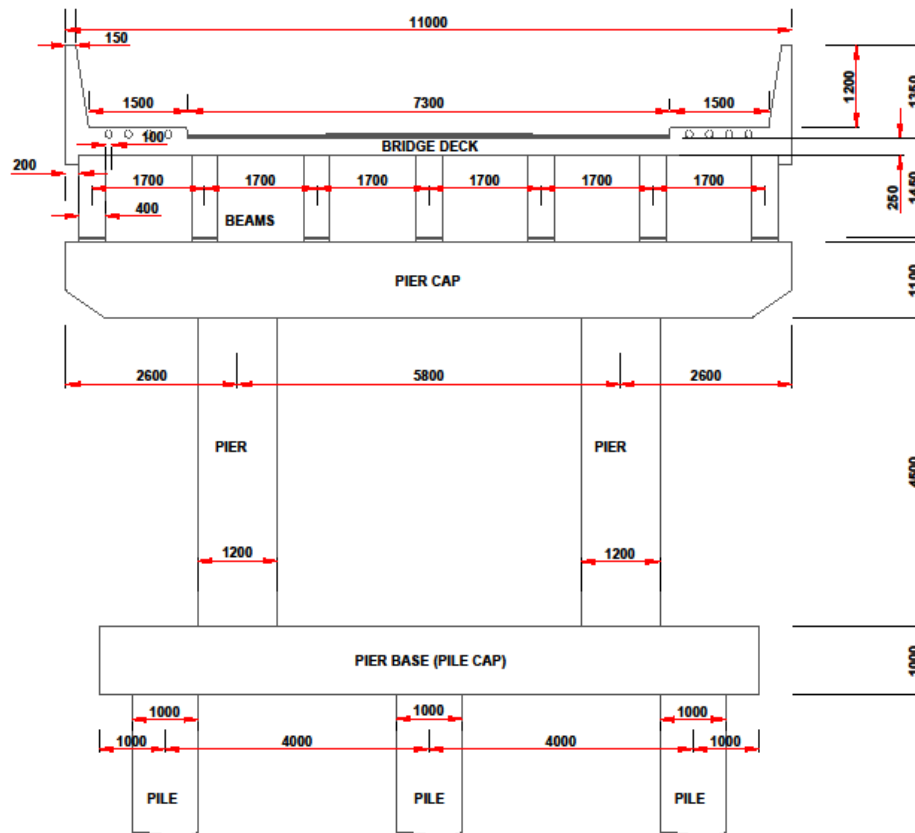


Figure 2. Typical cross section of the bridge deck.



3.2 Generation of Limit State Functions for Time-Variant Component Reliability Analysis.

The two-span bridge was analyzed with respect to the possible occurrence of 10 different failure modes. Each failure mode i is described by a limit state function $G(X)_i = 0$, such that $G(X)_i \leq 0$ defines the failure state and $G(X)_i > 0$ defines the safe state.

These include failure of the bridge deck in bending, moment and shear failure of the beams, and multiple failure modes of the pier cap, pier, abutment and foundations. Limit state equations in terms of the random variables were developed for the ten (10) failure modes for members in bending affected by chloride induced corrosion. The failure modes under investigation are enumerated in equations (12) to (21). Table 1 shows the statistical models of the basic design variables associated with failure mode 1 whose limit state equation is presented in equation 12.

Failure Mode 1: Failure of the Deck in Bending.

$$G(X)_1 = \phi_R A_s(t) f_{yk}(t) \left[d - \frac{A_s(t) f_{yk}(t)}{2b f_{ck}} \right] - [3.5\phi_G + [0.057Q_k(t) + 0.39q_k]\phi_Q] * 10^6 \quad (12)$$

Failure mode 2: failure of Interior Beam in Bending.

$$G(X)_2 = \phi_R A_s(t) f_{yk}(t) \left[d - \frac{A_s(t) f_{yk}(t)}{2b f_{ck}} \right] - [880.59\phi_G + (51.64q_k + 7.02Q_k)\phi_Q] * 10^6 \quad (13)$$

Failure mode 3: Failure of Exterior Beam in Bending.

$$G(X)_3 = \phi_R A_s(t) f_{yk}(t) \left[d - \frac{A_s(t) f_{yk}(t)}{2b f_{ck}} \right] - [816.47\phi_G - (30.38q_k + 7.02Q_k(t))\phi_Q] * 10^6 \quad (14)$$

Failure Mode 5: Failure due to Positive (sagging) Moment on the Pier Cap.

$$G(X)_5 = \phi_R A_s(t) f_{yk}(t) \left[d - \frac{A_s(t) f_{yk}(t)}{2b f_{ck}} \right] - [71.86\phi_G - 1.2Q_k\phi_Q + 16.34q_k\phi_Q] * 10^6 \quad (15)$$

Failure Mode 6: Negative (Hogging Moment) Moment on the Pier Cap.

$$G(X)_6 = \phi_R A_s(t) f_{yk}(t) \left[d - \frac{A_s(t) f_{yk}(t)}{2b f_{ck}} \right] - [784.66\phi_G + (6.55Q_k(t) + 23.39q_k)\phi_Q] * 10^6 \quad (16)$$

Failure Mode 7: Top of the Pier Crushing.

$$G(X)_7 = (0.8A_p f_{ck} + A_s(t) f_{yk}(t)) \phi_R - [1,111.11\phi_G + (7.57Q_k(t) + 41.93q_k)\phi_Q] * 10^3 \quad (17)$$



Failure Mode 8: Bottom of the Pier Crushing.

$$G(X)_8 = [0.8A_p f_{ck} + A_s(t) f_{yk}(t)] \phi_R - [1,282.88 \phi_G + (7.57 Q_k(t) + 41.93 q_k) \phi_Q] * 10^3 \quad (18)$$

Failure Mode 9: Failure of Pier Pile Cap in Bending.

$$G(X)_9 = \phi_R A_s(t) f_{yk}(t) \left[d - \frac{A_s(t) f_{yk}(t)}{2b f_{ck}} \right] - [239.62 \phi_G + (0.89 Q_k + 4.92 q_k) \phi_Q] * 10^6 \quad (19)$$

Failure Mode 10: Failure of Abutment Wall in Bending.

$$G(X)_{10} = \phi_R A_s(t) f_{yk}(t) \left[d - \frac{A_s(t) f_{yk}(t)}{2b f_{ck}} \right] - [(35.82 K_a \gamma_{bf} + 6.26) \phi_G + \{942.38 K_a + 186.77 + (0.025 Q_k + 0.32 q_k)\} \phi_Q] * 10^6 \quad (20)$$

Failure Mode 11: Failure of Abutment Base in Bending.

$$G(X)_{11} = \phi_R A_s(t) f_{yk}(t) \left[d - \frac{A_s(t) f_{yk}(t)}{2b f_{ck}} \right] - [(251.86 + 1.26 \gamma_{bf}) \phi_G + (0.08 Q_k + 0.98 q_k) \phi_Q] * 10^6 \quad (21)$$

Where;

$$A_s(t) = \frac{n\pi D(t)^2}{4}, \text{ Area of steel reinforcement at time } t.$$

$$Q_k(t) = Q_k(1 + \lambda_m)^t, \text{ Truck load at time } t.$$

$$f_{yk}(t) = \left(1 - \alpha_{yk} \frac{A_s(t)}{A_{so}}\right) f_{yko}, \text{ Design strength of reinforcement at time } t.$$

f_{yko} , Design strength of reinforcement at time of construction (age of 0 years), A_{so} , Area of steel reinforcement at time of construction (age 0 years), Q_k , Design truck axle load (100 kN per m), q_k , UDL traffic load (9.0 kN/m²), f_{ck} , Characteristics strength of concrete, γ_{bf} , Unit weight of backfill material, A_p , Cross sectional area of pier, b , Width of section, d , Effective depth of section, n , is the number of bars.



Table 1: Statistical model of the basic design variables for Failure mode1 (Failure of Deck in Bending).

S/N o.	Design variable	Notation	Unit	Distribution model	Mean	COV*	Source
1	Resistance Model Uncertainty	ϕ_R	-	Normal	1.0	0.05	[35]
2	Permanent Load Model Uncertainty	ϕ_G	-	Normal	1.05	0.10	[36]
3	Traffic Load Model Uncertainty	ϕ_{Qk}	-	Lognormal	1.0	0.18	[37]
4	Truck Traffic Load	$Q_k(t)$	kN	Normal	$100(1 + \lambda_m)^t$	0.18	[38]
5	UDL traffic load	q_k	kN/m ²	Normal	9.0	0.10	[38]
6	Concrete compressive strength	f_{ck}	N/mm ²	Lognormal	25	0.15	[39]



7	Steel strength at time t	$f_{yk}(t)$	N/mm ²	Lognormal	$\left(1 - 0.01 \frac{A_s(t)}{A_{so}}\right) 500$	0.10	[39]
8	Area of steel at time t	$A_s(t)$	mm ²	Normal	$\frac{5\pi D(t)^2}{4}$	0.024	[40]
9	Width of slab	b	mm	Normal	1000	0.05	[35]
10	Effective depth	d	mm	Normal	194	0.05	[35]

* COV: Coefficient of Variation.

Based on these limit state equations, the reliability index with respect to the occurrence of each possible failure mode was computed using the First Order Reliability Method approach. All random variables were transformed to uncorrelated standard normal variable and an iterative search technique using genetic algorithm was used to compute the reliability index β .

4.0 Results and Discussion

The study implemented structural reliability analysis for the investigation of the time variant reliability of reinforced concrete bridge components subjected to chlorides induced corrosion. The First Order Reliability Method (FORM) was used and the reliability indices were generated using the developed MATLAB program. The results are presented in Figures 3 to 11.



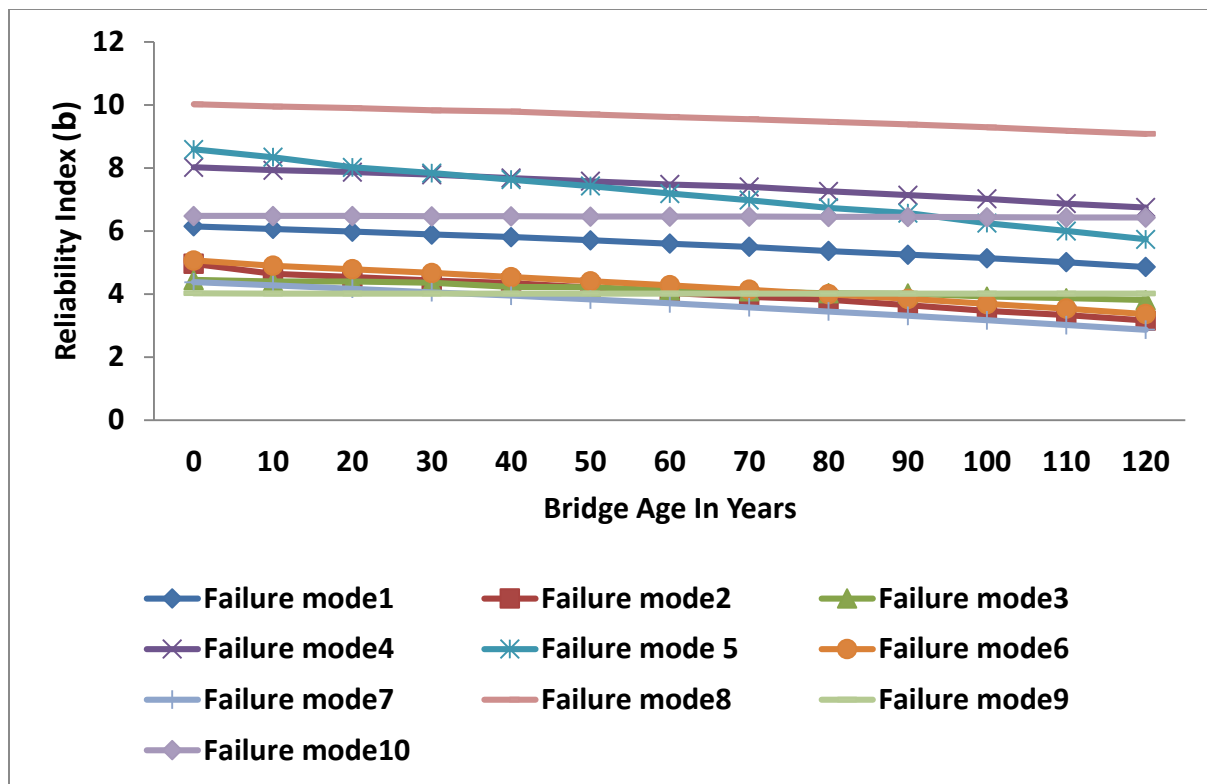


Figure 3: Variation of Safety Index against Bridge Age for corrosion rate of 0.00 mm/year.

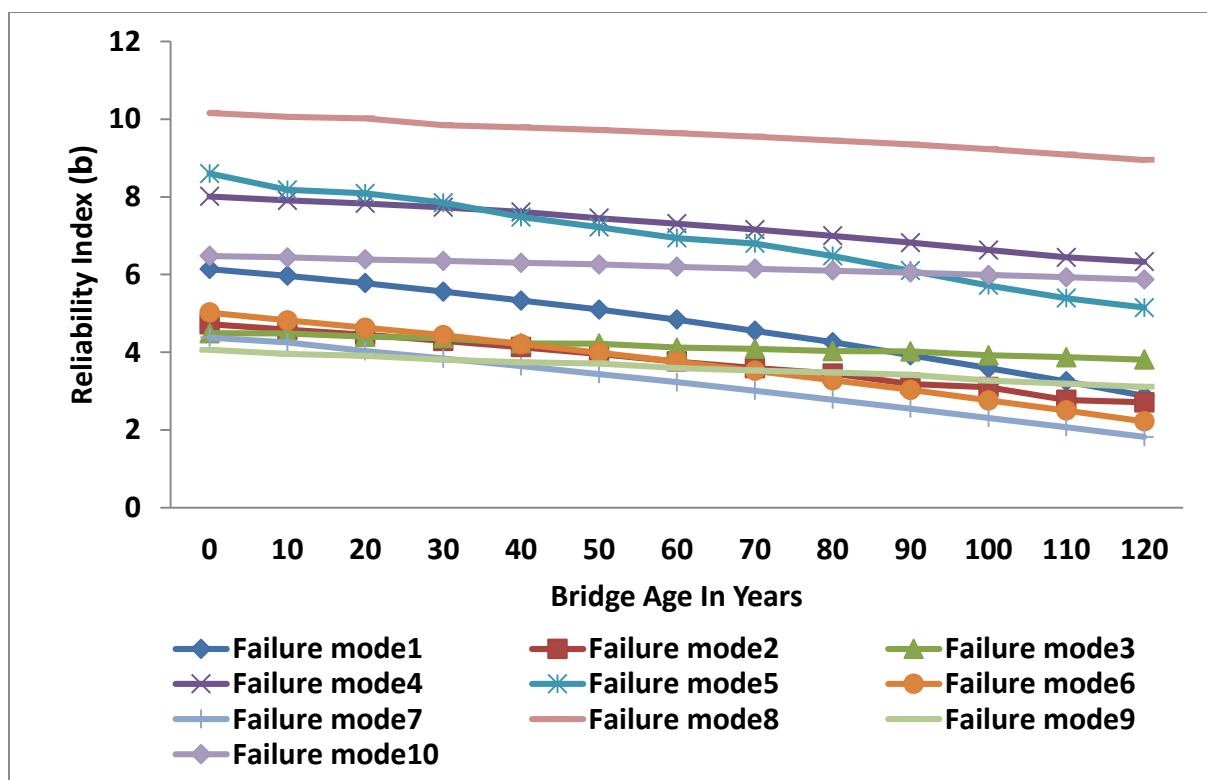


Figure 4: Variation of Safety Index against Bridge Age for corrosion rate of 0.02 mm/year.

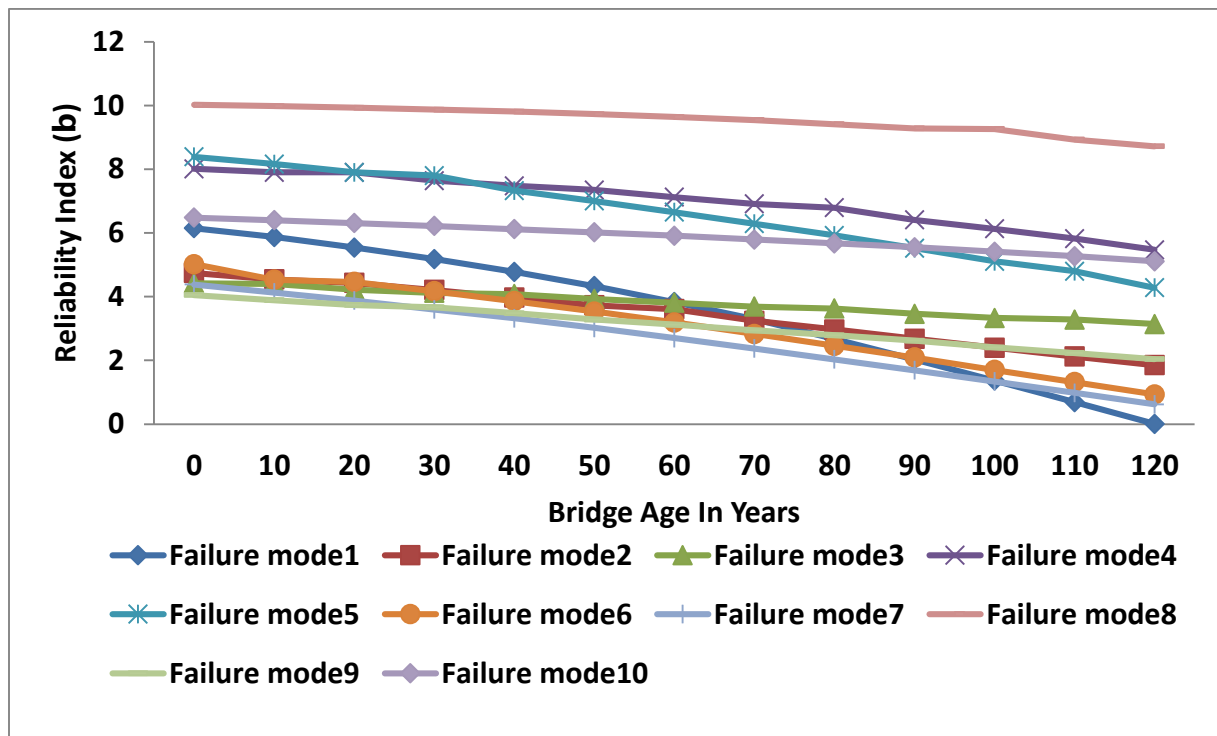


Figure 5: Variation of Safety Index against Bridge Age for corrosion rate of 0.04 mm/year.

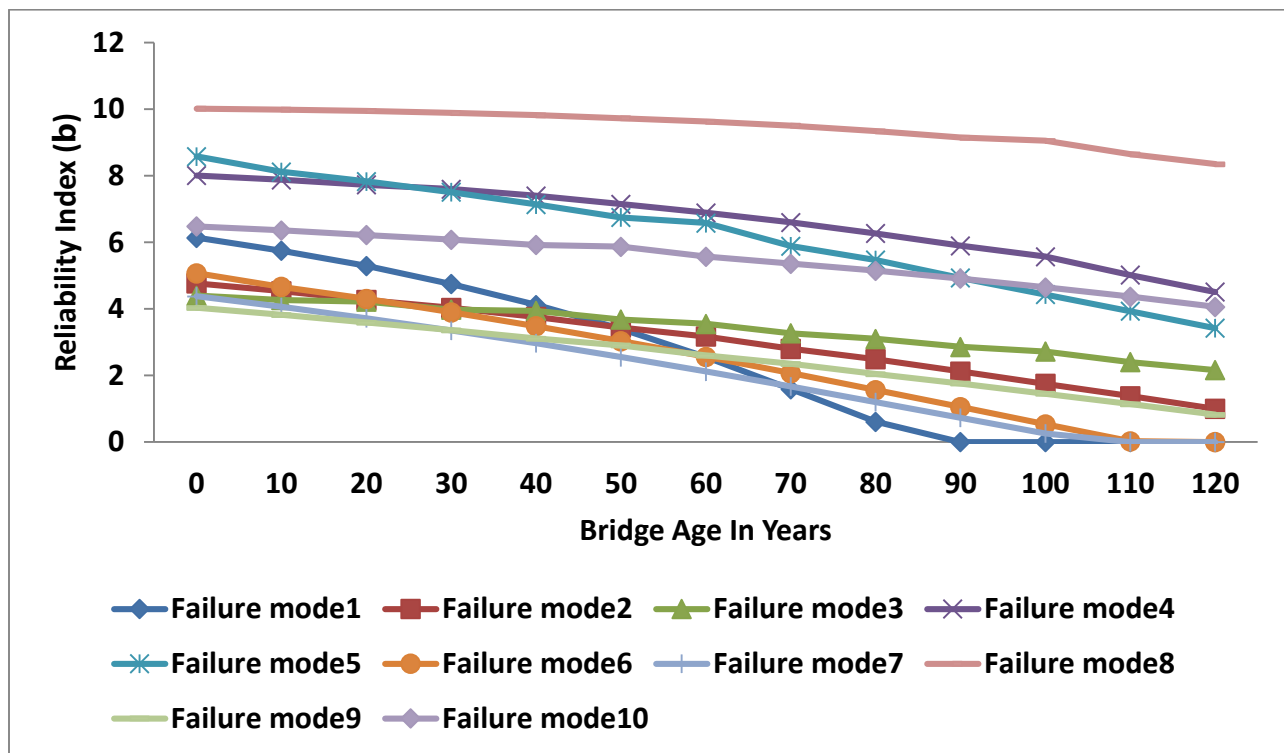


Figure 6: Variation of Safety Index against Bridge Age for corrosion rate of 0.06 mm/year.

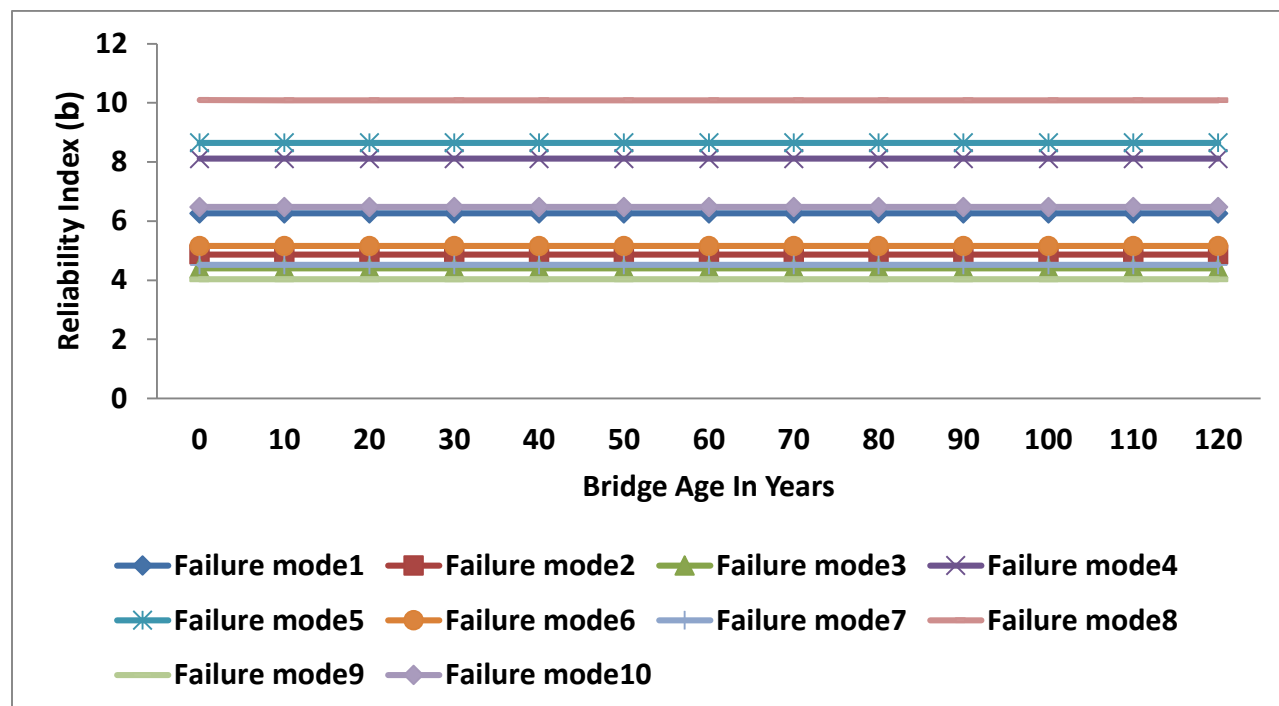


Figure 7: Variation of Safety Index against Bridge Age for corrosion rate of 0.00 mm/year without load growth.

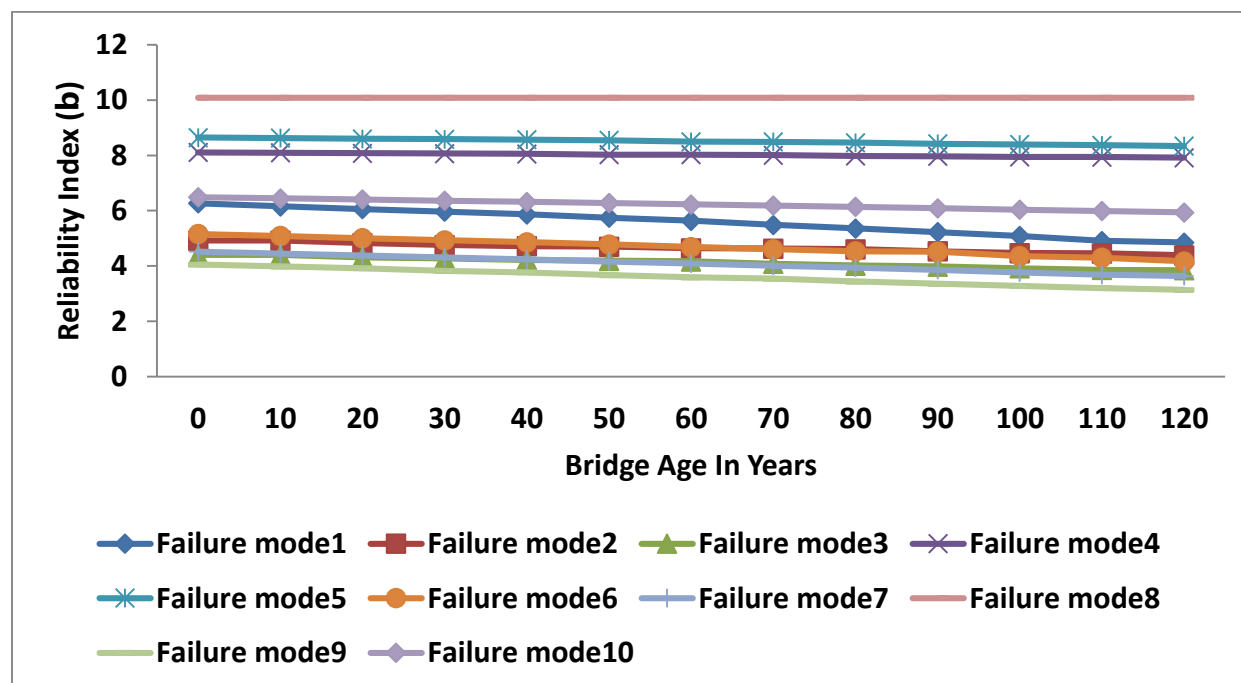


Figure 8: Variation of Safety Index against Bridge Age for corrosion rate of 0.02 mm/year but without load growth.

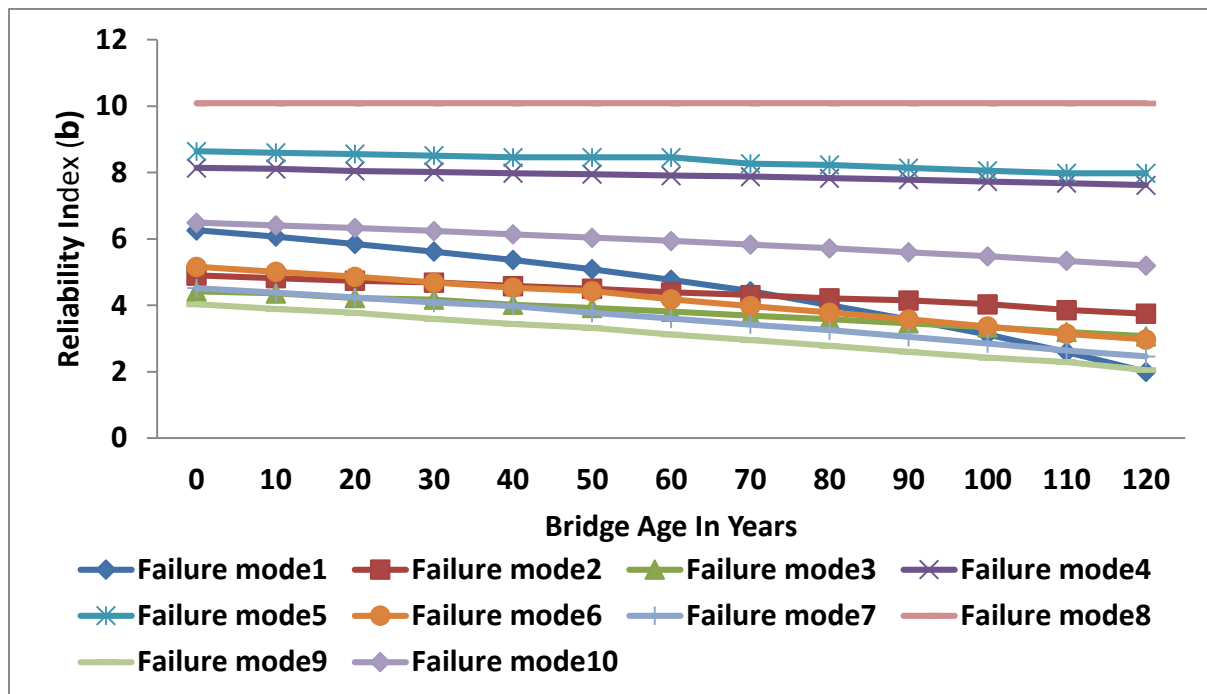


Figure 9: Variation of Safety Index against Bridge Age for corrosion rate of 0.04 mm/year but without load growth.



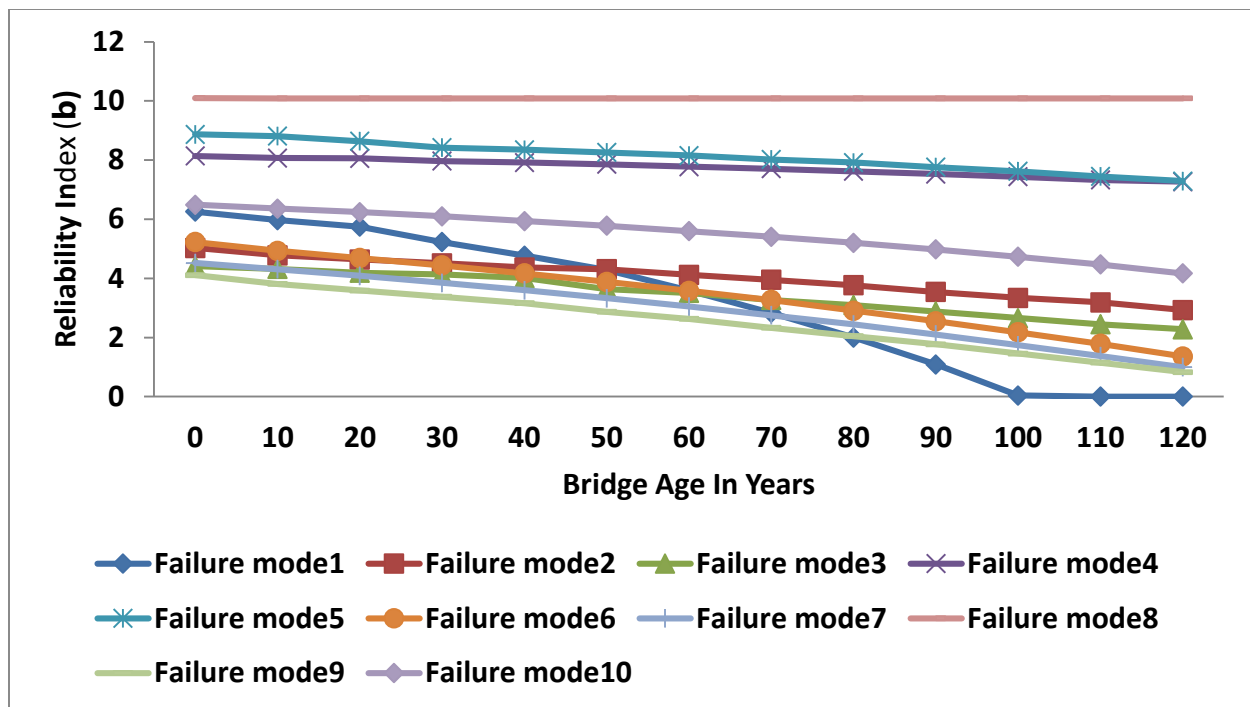


Figure 10: Variation of Safety Index against Bridge Age for corrosion rate of 0.06 mm/year but without load growth.

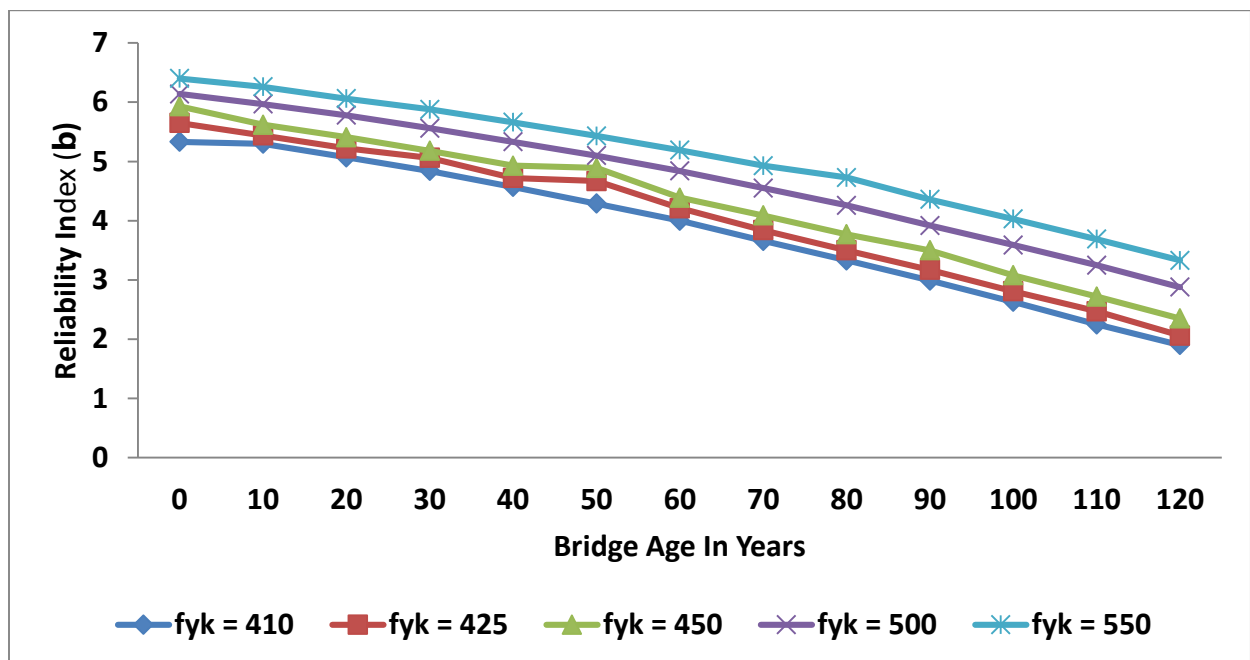


Figure 11: Variation of Safety Index against Bridge Age for Failure mode1 at various strength of steel reinforcement.



Figures 3 to 6 show the relationship between reliability index and bridge age for various corrosion rate scenarios of 0.00, 0.02, 0.04 and 0.06 mm/year with a truck load growth rate of 0.005. As the corrosion initiates (when the corrosion initiation time T_i in years is greater than zero), the structure is now exposed to corrosion. As the exposure time increases the capacity of the components to resist applied loading decreases. The extent to which the load capacity is lost depends on the corrosion rate, γ_{corr} and truck load growth λ_m . It is observed from the plots (Figures 4 to 7) that the reliability index of the components exposed to corrosion decrease linearly with the corrosion exposure time (bridge age).

Figure 3 shows a typical scenario of the structure subject to only truck load growth, but with no exposure to corrosion ($\gamma_{corr} = 0.00$). A look at the curves shows that Failure mode2 indicates the highest capacity loss (36.29%) over the exposure time of 120 years. It is closely followed by Failure mode7, Failure mode6, Failure mode5, Failure mode1, Failure mode4, Failure mode3, Failure mode8, Failure mode10, and Failure mode9 with capacity losses of 34.47%, 33.53%, 33.18%, 20.98%, 15.84%, 14.38%, 9.38%, 0.77% and 0.25% respectively over the exposure period of 120 years.

As the corrosion rate increases to 0.02, 0.04 and 0.06 mm/year with incremental rate of the design load (growth rate) still 0.005 (Figure 4 to 6). Failure mode 1 experiences the highest capacity loss, dropping by 53.09%, 99.88% and 100% respectively. The other failure modes with their corresponding capacity losses are: Failure mode7: 58.54%, 85.84% and 100%; Failure mode6: 55.78%, 81.47% and 100%; Failure mode2: 42.58%, 60.97% and 79.10%; Failure mode9: 23.40%, 49.63% and 79.65%; Failure mode5: 40.12%, 48.99% and 60.14%; Failure mode3: 15.14%, 28.96% and 50.91%; as well as Failure mode4: 20.97%, 31.71% and 43.70%. The least capacity losses are experienced by Failure mode8 with capacity losses of 11.91%, 12.97% and 16.67% and Failure mode10 with capacity losses of 9.41%, 21.14% and 37.35% respectively. This indicates the need for proper monitoring of those components responsible for the higher risk failure modes with a view of ensuring adequate preventive measures against capacity losses resulting from corrosion. This will reduce the risk of premature failure and ensure attainment of the design service life of the structure.



Figure 7 shows a situation in which the structure is subjected to an ideal design environment, where it experiences no corrosion and load growth. In this case there is no loss of capacity by all the failure modes. The reliability indices of all the failure modes also meets the target reliability index value of 4.02 prescribed for reinforced concrete bridges in the EN 1990 (2002) for a reference period of 120 years. This implies that all the components affected by these failure modes will maintain their full capacity. However, with exposure to time dependent corrosion and load the reliability index gradually drops with time.

With reference to Figure 3, only Failure mode1, Failure mode4, Failure mode5, Failure mode8, Failure mode9 and Failure mode10 meets the target reliability index value. As the corrosion rate increase to 0.02 and 0.04mm/year (Figures 4 and 5), only Failure mode4, Failure mode5, Failure mode8 and Failure mode10 meets the code requirement. Figures 6 also show that only Failure mode4, Failure mode8, and Failure mode10 satisfies the code requirement; implying that the higher the corrosion rate, the higher the loss of structural capacity.

Figure 8 to 10 show the variation of safety index against bridge age at the initial design load (no load increment/growth) but varying corrosion rates of 0.02, 0.04 and 0.06mm/year. Using the most corrosive environment as a reference (Figure 10), it is observed that the capacity loss is lower when compared with a similar situation but with load growth (Figure 6). The capacity losses for Failure mode1, Failure mode2, Failure mode3, Failure mode4, Failure mode5, Failure mode6, Failure mode7, Failure mode8, Failure mode9 and Failure mode10 are 100%, 41.75%, 48.42%, 10.69%, 17.81%, 74%, 77.88%, 0.10%, 79.81% and 35.75% respectively for corrosion exposure without load growth; and 100%, 79.10%, 50.91%, 43.70%, 60.14%, 100%, 100%, 16.67%, 79.65% and 37.35% respectively for corrosion exposure with load growth. The only exception here is Failure mode9 whose capacity loss is slightly higher in the former condition. This clearly shows that time dependent corrosion and load has significant effects on reinforced concrete bridges. Once corrosion is initiated, the load carrying capacity continue to decrease with time and will be lost if adequate preventive measures are not taken. This will cause serious threat to life and properties as well as an increase in user cost.

Figure 11 displays the relationship between reliability index and characteristics strength of steel reinforcement for Failure mode1 over the bridge service life. It is clear from the plot that the



reliability index increase with time as the characteristics strength of steel increases. The characteristics strength of steel was varied from 410, 425, 450, 500 and 550N/mm² with the corrosion rate kept constant at 0.02mm/year and load growth rate of 0.005. It is observed from the plots that the reliability indices increase by an average value of 4.68% at age 0 years and by an average value of 15.17% at age 120 years as the characteristics strength of steel increase from 410 to 550N/mm². This implied that high strength steel increases the safety margin and hence the carrying capacity of reinforced concrete bridge structures subjected to corrosion and increment in the designed load.

4. Conclusion and Recommendations

This study used the probabilistic method to assess the component reliability of a reinforced concrete bridge exposed to chloride induced corrosion and load growth. Mathematical models for strength capacity loss and increment in design load (load growth) reported in literature were used in the derivation of the limit state functions. Ten failure modes for members in bending were considered to accommodate the time dependent effects in the structural reliability analysis.

First Order Reliability algorithms were used in generating the reliability indices in conjunction with the evaluated limit state functions. The developed algorithm was coded using MATLAB-based program, and the process was fully automated. The reliability of the components before and after the onset of corrosion was checked over the service life of the reinforced concrete bridge (120 years). It was shown that the load capacity loss ranges between 9.41% for the least deteriorated member's failure mode to 100% for the most deteriorated member's failure mode as the corrosion rate increases from 0.02 to 0.06 mm/year at load growth rate of 0.005.

Also the effects of perturbations in exposure to corrosive environment, increment in design load (load growth), and characteristics strength of steel showed that Lower capacity losses were obtained when the structure was subjected to a constant design load (growth rate = 0.00), with losses ranging from 0.25% for the least deteriorated member's failure mode to 34.47% for the most deteriorated member's failure mode as the corrosion rate increases from 0.02 to 0.06 mm/year. The load capacity however increased by an average value of 4.68% at age 0 years and by an average value of 15.17% at age 120 years as the characteristics strength of steel increase from 410 to 550N/mm².



The study also found that corrosion of reinforced concrete bridges exposed to chloride ingress is a very serious durability issue, especially when accompanied by increment in the design load (load growth). Therefore the effect of chloride ingress on reinforcement cannot be overlooked, there is need to employ preventive maintenance measures either before the onset of corrosion or before it propagates to unfavourable levels so as to ensure adequate structural performance and also prevent premature failure.

Secondly, since the bridge has a design service life with decreasing structural capacities during service; there is the need to guarantee its safe function using appropriate planned preventive maintenance programs over the service life of the bridge.

REFERENCES

- Adamu, L., Mohammed, J. K., Jaafar, A. S. and Ibrahim, A. Probabilistic Durability Analysis of Reinforced Concrete Members with Corroded Reinforcing Steel. *Jordan Journal of Civil Engineering*, Vol. 8, No. 3, 2014, pp. 303-311
- Almusallam, A. A. Effect of Degree of corrosion on the Properties of Reinforcing Steel bars. *Construction and Building material*, Vol. 15, 2001, pp. 361 – 368.
- Ang, A. H. S and Tang, W. H. *Probability Concept in Engineering*. 2nd Edition. John Wiley & Sons, Hoboken, NJ, USA, 2007.
- Bertolini, L., Elsener, B., Pedferri, P. and Polder, R *Corrosion of Steel in Concrete*. Wiley-VCH, Weinheim, Germany, 2004.
- Bigaud, D. Ali, O. Charki, A. Time Variant Flexural Reliability of Reinforced Concrete Highway Bridges Strengthened by Means of Carbon Fiber Reinforced Polymer. *International Journal of Engineering Science and Innovative Technology (IJESIT)*, Vol. 3 issue 6, 2014.
- Biondini, F. and Frangopol, D. M. Time Effect on Robustness and Redundancy of Deteriorating Concrete, *11th International Conference on Structural Safety and Reliability (ICOSSAR 2013)*, New York, NY, USA, June 16-20, 2013. In: *Safety, Reliability, Risk and Life-cycle Performance of Structures and Infrastructures*, G. Deodatis, B. R. Ellingwood, D. M. Frangopol (Eds), CRC Press/Balkema, Taylor & Francis Group, London, UK.
- Biondini, F. and Frangopol, D. M. Life-Cycle Performance of Structural Systems: A Review. *Journal of Structural Engineering*, ASCE, 2014.
- Bordallo-Ruiz, A., McNally, C., Caprani, C. C., Colin, C. and O'Brien, E. J. The Structural Reliability of Bridges Subject to Time-Dependent Deterioration B.H.V. Topping. (ed.). *Proceedings of the Eleventh International Conference on Civil, Structural*
- Cady, P. D. and Weyers, R. E. Deterioration Rates of Concrete Bridge Decks. *Journal of Transportation Engineering*, ASCE Vol. 110, No. 1, 1984, pp. 34-44.
- Cairns, J., Plizzari, G. A., Du, Y., Law, D. W. and Franzoni, C. Mechanical Properties of Corrosion Damaged Reinforcement *ACI Material Journal*, Vol. 102, 2005 pp. 256-264.



- DNV-OS-C50 (2010). Offshore Concrete structures. <http://www.dnv.com> retrieved on Monday 10th August, 2012. and *Environmental Engineering Computing*. Civil-Comp Press, 2007.
- EN 1992-1-1. *Eurocode 2 Design of concrete Structures. Part 1-1 General Rules and Rules for Buildings*, CEN Brussels, 2004.
- EN 1992-2. *Eurocode 2 Design of concrete Structures. Part 2 Concrete Bridges. Design and Detailing Rule*, CEN Brussels, 2005.
- EN 1991-1-1. *Eurocode 1 Actions on Structures. Part 1-1 General Actions. Densities, Self-Weight, Imposed loads for Buildings*, CEN Brussels, 2002.
- EN 1991-1-5. *Eurocode 1 Actions on Structures. Part 1-5 General Actions. Thermal Actions*, CEN Brussels, 2004.
- EN 1991-2. *Eurocode 1 Actions on Structures. Part 2 Traffic Loads on Bridges*, CEN Brussels, 2003.
- EN 1997-1. *Eurocode 7 geotechnical Design. Part 1. General Rules*. CEN Brussels, 2004.
- EN 1990. *Eurocode-Basis of Structural Design*, CEN Brussels, 2002.
- Estes A. C. and Frangopol D. M. Repair Optimization of Highway Bridges Using System Reliability Approach. *Journal of Structural Engineering*, Vol. 125, issue 7, 1999.
- Frangopol, D. M. and Ellingwood, B. R. Life-Cycle Performance, Safety, Reliability and Risk of Structural Systems, Editorial, *Structure Magazine*, Joint Publication of NCSEA, CASE, SEI, 2010.
- Frangopol D.M and Hendawi, S Incorporation of Corrosion Effects in Reliability Based Optimization of Composite Hybrid Plate Girders. *Safety*, Vol. 16, Nos. 1& 2, 1994, pp. 145-169.
- Frangopol D. M and Moses F. Reliability Based Structural Optimization. *Advances in Design Optimization*. H. Adeli ed., Chapman and Hall Ltd., London, England, 1994, pp. 492-570.
- Hasofer, A. M. and Lind, N. C. An Exact and Invariant First-Order Reliability Format. *Journal of Engineering Mechanics*, ASCE Vol.100, 1974, pp. 111-121.
- Kupwade-Patil, K., Cardenas, H., Gordon, K. and Lee, L. Corrosion Initiation in RC Beams Via Nanoparticles Treatment. *ACI materials Journal*, Vol. 109, No. 6, 2012, pp. 617-626.
- Li, C., Melchers, R. E. and Zheng, J. Analytical Model for Corrosion-Induced Crack Width in Reinforced Concrete Structures, *ACI Structural Journal*, Vol. 103, No. 4, 2006, pp. 479-487.
- Liu, T. and Weyers, R. W. Modeling the Dynamic Corrosion Process in Chloride Contaminated Concrete Structures, *Cement and Concrete Research*, Vol. 28, No. 3, 1998, pp. 365-379.
- Li, C. Q. Life Cycle Modeling of Corrosion Affected Concrete Structures Initiation. *Journal of Materials in Civil Engineering*, Vol. 15, No. 6, 2005, pp. 594-601.
- Mori, Y. and Ellingwood, B. R. Reliability-Based Service-Life Assessment of Aging Concrete Structures, *Journal of Structural Engineering*, Vol. 119, No. 5, 1993, pp. 1600-1621.
- Sarja, A. Towards Practical Durability Design of Concrete Structures, *Proc. of 7DBMC 7th International Conference on Durability of Building Materials and Components*. Stockholm, Sweden, E & FN spon, London, UK, EU. 2 1238-1247, 1996.



- Tarighat, A. and Jalalifar, E. Assessing the Performance of Corroding RC Bridge Decks: A Critical Review of Corrosion Propagation Models, *Civil Engineering Infrastructures Journal*, 2013. University of Tehran Publishing.
- Thoft-Christensen, P. Stochastic Modeling of the Crack Initiation Time for Reinforced Concrete Structures. *2000 Structures Congress*, Philadelphia, May 8-10, 2000.
- Vu, K. A. T and Stewart, M. G. Predicting the Likelihood and Extent of Reinforced Concrete Corrosion Induced Cracking. *Journal of Structural engineering*, ASCE, Vol. 131, 2005, pp. 1681-1689.
- Vu, K. A. T and Stewart, M. G, Structural Reliability of Concrete Bridges Including Improved Chloride-Induced Corrosion Models, *Structural Safety*, Vol. 22, No. 4, 2000, pp. 313-333.

