Full Paper

SYSTEM-BASED RELIABILITY ANALYSIS OF THE PREVENTIVE MAINTENANCE STRATEGIES OF A REINFORCED CONCRETE BRIDGE

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ABSTRACT

A system-based time-dependent reliability analysis was carried out over the service life of a reinforced concrete bridge using a MATLAB program, BRELA (BRidge RELiability Analysis). Component reliability analysis for the bridge members was evaluated using a built-in MATLAB based sub program in BRELA and the system reliability index "β_{sys}" calculated over time using the system reliability analysis component of the main program. The bridge was modelled as a hybrid/combined system (series-parallel subsystems). Nineteen (19) failure modes for the bridge components were identified and their limit state functions incorporating load growth and deterioration models of the steel reinforcement due to corrosion were developed. The result of the system reliability analysis shows a system performance loss of 25.65–100% as corrosion rate increased from 0–0.06 mm/year for a load growth rate of 0.5%. Considering several realistic preventive maintenance options and their associated costs, the reinforced concrete bridge is maintained by ensuring that the commencement of preventive maintenance measures is a function of the deterioration rate and the target performance level. For a target system reliability index of 3.0 with a load growth of 0.5%, preventive measures have to be implemented around 30, 25 and 20 years after construction for corrosion rates of 0.02, 0.04 and 0.06 mm/year respectively.

Keywords: Concrete bridge, Corrosion rate, Preventive maintenance, System reliability index.

1. INTRODUCTION

Bridge structures in Nigeria and other part of the world undergo structural deterioration with time due to a range of causes one of which is corrosion. Environmental and loading conditions may cause changes in structural strength and stiffness, impairing the safety and serviceability of bridges (Mori and Ellingwood, 1993; O' Brien *et al.*, 2014; Li *et al.*, 2015). The economic impact of structural deterioration particularly for concrete bridges exposed to corrosion, is relevant and emphasizes the importance of maintenance and repair interventions to avoid or reduce structural deficiencies (NCHRP, 2006; ASCE, 2013; Biondini and Frangopol, 2014).

As a consequence, for deteriorating structures the required level of safety and serviceability should be ensured not only at the initial time, but over the expected service lives (Frangopol and Ellingwood, 2010; Frangopol, 2011; Biondini and Frangopol, 2014). A rational approach to design, assessment, maintenance and repair of deterioration in bridges requires a modelling of the structural system over the entire life-cycle by taking into account the effects of deterioration processes, time-variant loadings, maintenance actions and repair interventions (Frangopol, 2011). The closure due to malfunction of a bridge will represent a serviceability failure resulting in major financial problems both locally and nationally (Tantele and Onoufriou, 2006). Bridges cannot last in perpetuity but adequate maintenance can help secure their maximum service life.

The integration of structural performance assessment methods and time-dependent reliability analysis techniques has tremendous potential in providing cost-effective maintenance strategy for aging structures (Nepal and Chen, 2015). Cole (2000) observed that, whilst the cost of maintenance is increasing constantly, funding is generally inadequate to allow indiscriminate repair of the entire bridge network in any given funding year.

Two types of maintenance work have been enumerated by Das (1999); preventive maintenance which if not done, will cost more at a later stage to keep the structures in a safe condition, and essential maintenance which is required to keep the structures safe. Preventive maintenance is a broad term that encompasses a set of activities aimed at improving the overall reliability and availability of a system. Das (1999) and Frangopol *et al.*, (2000) have therefore suggested that preventive maintenance can help reduce the cost of maintenance required for the bridge useful life. Preventive maintenance can both help postpone essential rehabilitation works, and extend the service life of bridges in a cost-effective manner.

The structural performance of reinforced concrete structures such as bridges is time dependent due to the damaging process induced by environmental and loading conditions. The live load represented by the weight of truck traffic 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). This eventually affects the service life of the concrete structures and also increases the cost of maintenance and rehabilitation over time. Thus, maintenance and management of corrosion affected reinforced concrete have become worldwide issues for engineers and asset managers (Tilly and Jacobs, 2007; Papkonstantinou and Shinozuka, 2013; Chen and Alani, 2013; Nepal and Chen, 2015). There is therefore a need for a technique which can confidently predict how these deteriorating structures will respond to the deterioration, when they will be unsafe to deliver the required capacity, and what actions will be required to ensure their continual performance.

The highways department of the Federal Ministry of Power, Works and Housing carried out an inventory of bridges in 2016, which shows a total of about 1,745 bridges within the Nigerian federal highway network. Most of these bridges are over 50 years old and were not designed to meet the current network demand. These bridges are at various stages of deterioration due to a combination of several factors such as: insufficient or lack of maintenance/repairs, vandalizing, accidental damage, environmental effects, increasing traffic loads and intensities among others. Keeping these assets in a functional shape requires large expenditure which due to scarcity of resources and funds is very difficult.

Prevention, they say is better than cure and is particularly true for bridges that, due to detrimental factors their safe and efficient operation may at some point in future be endangered, if action is not taken now (Tantele, 2005). Deterioration of highway structures progresses at an increasing rate with time resulting in safety concerns and increased total life-cycle cost. These effects can be taken care of by applying preventive maintenance in the early state of deterioration than postponing it until the deficiencies become evident.

Considering all of the aforementioned, it is essential that the system reliability of a bridge structure be investigated so that a better idea of the safety of the whole structure as a system can be obtained. This will help in identifying the components which are significant to keeping the system safe. It will also guide in identifying the appropriate preventive maintenance measures and the best sequence of application on the critical components in the system in each period over a planning period. This will minimize the overall costs subject to a constraint on reliability or maximize the reliability of the system subject to a constraint on the budget.

This study developed a system-based reliability analysis of the preventive maintenance strategies for reinforced concrete bridges over their service life using time-dependent techniques. This was achieved by identifying the relevant failure modes of a typical reinforced concrete bridge, developing limit state functions with respect to the occurrence of each possible failure mode taking into account deterioration models that describes how the structural performance within a given environment is expected to change over time. The safety levels with respect to the occurrence of each possible failure mode over time were computed and the safety level of the components of the bridge and its overall system safety level was determined to a specified system target safety level. The target system reliability index was used to establish preventive maintenance criteria and the optimum preventive maintenance strategy was developed.

2. LIMIT STATE FUNCTIONS FOR A REINFORCED CONCRETE BRIDGE

The existing reliability models for bridge structures is mainly associated with the ultimate limits states (ULS), mostly related to the bending capacity, shear capacity and stability (Ghodoosipoor, 2013). The serviceability limit states (SLS) may be involved when the target is users' comfort (Nowak, 2004). The focus of this study is on the ultimate limit state. In order to carry out reliability analysis of bridge components, there is need to develop limit state functions. Each limit state is associated with a set of limit state functions which determines the boundaries of acceptable performance (Nowak and Zhou, 1990). Basic limit state functions are always in the form of Equation (1).

$$Z = g(R,S) = R - S \text{ or } g(R,S) = \frac{R}{S} - 1 \qquad (1)$$

Where Z is the performance function, R represents the resistance (or capacity) and S represents the load effect (or demand). Setting the border of $g(R,S) = 0$ between acceptable and unacceptable performance, the limit state function of $g(R,S) > 0$ represents the safe performance and $g(R,S) < 0$ represents failure. Specialized limit state functions are formulated in details for each bridge element (Wang, 2012). In some cases, there may be more than one failure mode for a bridge element. The development of limit state functions starts from the selection of essential failure modes which determines structural performances. The essential failure modes considered in this study for the elements of a typical bridge are listed in Table 1.

Table 1: Critical Failure Modes for Typical Bridge Elements (Source: (Wang, 2012))

Structural Elements	Failure Modes Considered
Deck	Moment/Flexure.
Beams	Moment, Shear.
Bearings	Crushing.
Piers Cap	Shear, Positive Flexure, Negative Flexure.
Piers	Top pier-crushing, Bottom pier-crushing.
Abutments	Overturning, Moment, Shear, Sliding, and Bearing.
Pile cap	Shear, Moment, and Bearing.

2.1. Reliability Index:

Probabilistic methods used in structural design are based on the reliability index β which is related to the probability of failure P_f by;

$$\beta = -\Phi^{-1}(P_f) \qquad (2)$$

Where Φ^{-1} is the inverse standard normal distribution function.

Generally, if R and S are uncorrelated random variables, the reliability index can be calculated from (Hasofer and Lind, 1974),

$$\beta = \frac{\mu_R - \mu_S}{\sqrt{(\sigma_R^2 + \sigma_S^2)}} \qquad (3)$$

Where μ_R and μ_S are the mean values of R and S, and σ_R and σ_S are the standard deviation of R and S, respectively.

2.2. System Reliability:

In the preceding section, the reliability index was limited to the failure of one component according to one limit state function. A system can be defined as an assemblage of several components that serves some function or purpose (Ayyub and McCuen, 1997). In general, a component can fail in one of several failure modes. To treat these multiple failure modes, the component behaviour has to be modelled as a system. Failure of a single component (Series system) may cause failure of the entire system or the system may have redundancies where multiple components must fail (Parallel system) for the system to fail. For this study a hybrid system consisting of combinations of series and parallel subsystems is considered for the bridge system.

2.3. Target Reliability Levels:

The target reliability levels recommended in EN 1990 (2002) is adopted for the study which are related to consequences of failure that are primarily intended for new structures (Sykora *et al.*, 2011). These target reliability levels are recommended for two reference periods (1 and 50 years) based on three consequence classes (CC) in Annex B of EN 1990 (2002).

2.4. Structural Deterioration Models:

Deterioration models are used in predicting the state of a bridge element or structure over time so as to determine their maintenance needs. They can either be deterministic or probabilistic in nature (James *et al.*, 1991). Deterioration models for major distress mechanisms in reinforced concrete structures such as alkali-silica reaction, chloride induced corrosion of reinforcement are investigated by laboratory tests, statistical analysis and mathematical modelling (Gonzalez *et al.*, 1995; Leira and Lindgard, 2000; McGee, 2000; Papadakis *et al.*, 1996; Patev *et al.*, 2000 and Rendell *et al.*, 2002).

3. BRIDGE MODEL

A simply supported reinforced concrete bridge consisting of two equal spans of 15.0 m each which covers an effective length of 30.0 m located in an open area is considered in this study. The bridge superstructure, composed of seven 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 0.25m deep in-situ deck slab. 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. The bridge cross section used in this study is given in Figure 1.

The material properties were chosen according to EN 1992-1 (2004) and EN 1992-2 (2005), the strength classes of structural materials are $f_{ck} = \mathrm{C25/30}$ for concrete and $f_{yk} = \mathrm{B500C}$ for steel reinforcement. Density of reinforced concrete, γ_{conc} = 25.0 kN/m^3; density of asphalt concrete γ_{asph} = 23 kN/m^3 and weight of parapet wall = 0.5 kN/m.

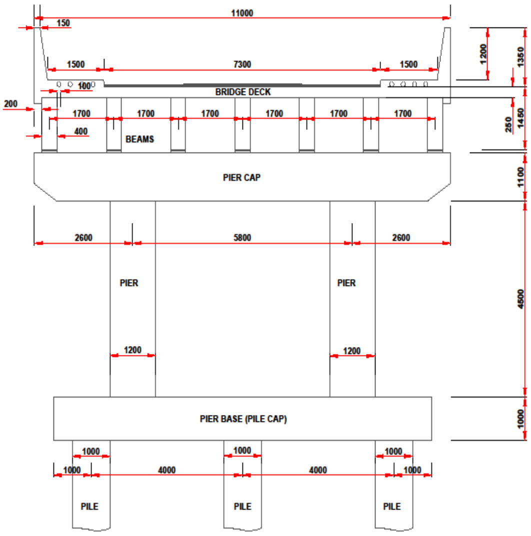

Figure 1: Typical bridge section

4. GENERATION OF LIMIT STATE FUNCTIONS FOR TIME- VARIANT COMPONENT RELIABILITY ANALYSIS

The capacity of a component is a function of its limit state under consideration (flexure, shear, crushing, overturning, etc.). The time-invariant limit state functions are modified using the time dependent random variables earlier discussed to form the time dependent (variant) reliability case. These are:

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

Diameter of reinforcement bar at time t:

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

Truck load at time t:

$Q_k(t) = Q_k(1 + \lambda_m)^t$ (6)

Design strength of reinforcement:

$f_{yk}(t) = \left(1 - \alpha_{yk}\frac{A_s(t)}{A_{so}}\right) f_{yko}$ (7)

The resulting time dependent (variant) limit state functions are now used in calculating the time variant reliability (safety) indices. Nineteen failure modes were considered and their corresponding limit state function are presented in Table 2.

Table 2: Failure Modes for the Typical Bridge and the Limit State Functions

Failure modes	Description
1	Failure of the Deck in Bending
$G(X)_1 = \phi_R A_s(t) f_{yk}(t)\left[d - \frac{A_s(t) f_{yk}(t)}{2bf_{ck}}\right] - 3.5\phi_G - [0.057Q_k(t) + 0.39q_k]\phi_Q * 10^6$	
2	Failure of Interior Beam in Shear.
$G(X)_2 = 0.138bdf_{ck}(1 - 0.004f_{ck})\phi_R - [234.83\phi_G + [2.02Q_k + 13.77q_k]\phi_Q] * 10^3$	
3	Failure of Interior Beam in Bending
$G(X)_3 = \phi_R A_s(t) f_{yk}(t)\left[d - \frac{A_s(t) f_{yk}(t)}{2bf_{ck}}\right] - [880.59\phi_G + (51.64q_k + 7.02Q_k)\phi_Q] * 10^6$	
4	Failure of Exterior Beam in Bending.
$G(X)_4 = \phi_R A_s(t) f_{yk}(t)\left[d - \frac{A_s(t) f_{yk}(t)}{2bf_{ck}}\right] - [816.47\phi_G - (30.38q_k + 7.02Q_k(t))\phi_Q] * 10^6$	
5	Failure of Exterior Beam in Shear.
$G(X)_5 = 0.138f_{ck}(1 - 0.004f_{ck})bd\phi_R - [217.73\phi_G + (2.52Q_k + 7.5q_k)\phi_Q] * 10^3$	
6	Failure of Pier Cap due to Shear
$G(X)_6 = 0.138f_{ck}(1 - 0.004f_{ck})\phi_R bd - [1{,}111.11\phi_G + (7.57Q_k + 41.93q_k)\phi_Q] * 10^3$	
7	Failure due to Positive (Sagging) Moment on the Pier Cap.
$G(X)_7 = \phi_R A_s(t) f_{yk}(t)\left[d - \frac{A_s(t) f_{yk}(t)}{2bf_{ck}}\right] - [71.86\phi_G - 1.2Q_k\phi_Q + 16.34q_k\phi_q] * 10^6$	
8	Negative (Hogging Moment) Moment on the Pier Cap.

$G(X)_8 = \phi_R A_s(t) f_{yk}(t)\left[d - \frac{A_s(t) f_{yk}(t)}{2bf_{ck}}\right] - [784.66\phi_G + (6.55Q_k(t) + 23.39q_k)\phi_Q] * 10^6$	
9	Top of the Pier Crushing
$G(X)_9 = \left(0.8A_p f_{ck} + A_s(t) f_{yk}(t)\right)\phi_R - [1{,}111.11\phi_G + (7.57Q_k(t) + 41.93q_k)\phi_Q] * 10^3$	
10	Bottom of the Pier Crushing
$G(X)_{10} = [0.8A_p f_{ck} + A_s(t) f_{yk}(t)]\phi_R - [1{,}282.88\phi_G + (7.57Q_k(t) + 41.93q_k)\phi_Q] * 10^3$	
11	Failure of Pier Pile Cap in Bending
$G(X)_{11} = \phi_R A_s(t) f_{yk}(t)\left[d - \frac{A_s(t) f_{yk}(t)}{2bf_{ck}}\right] - [239.62\phi_G + (0.89Q_k + 4.92q_k)\phi_Q] * 10^6$	
12	Failure of Pier Pile Cap in shear
$G(X)_{12} = 0.138f_{ck}(1 - 0.004f_{ck})\phi_R bd - [1{,}633.80\phi_G + (6.06Q_k + 33.54q_k)\phi_Q] * 10^3$	
13	Failure of Pier Pile Group
$G(X)_{13} = 9[(L_b + W_b)H_b C_s + C_b L_b W_b]\phi_R - [5{,}940.00\phi_G + (15.14Q_k + 83.86q_k)\phi_Q]$	
14	Failure of Abutment due to Overturning
$G(X)_{14} = [(7{,}937.13 + 52.65\gamma_{bf})\phi_G + (1.62Q_k + 20.26q_k)\phi_Q] - [50.99\gamma_{bf}K_a\phi_G + (720.07K_a + 186.77)\phi_Q + 1.64h_w^3]$	
15	Failure of Abutment Wall in Bending.
$G(X)_{15} = \phi_R A_s(t) f_{yk}(t)\left[d - \frac{A_s(t) f_{yk}(t)}{2bf_{ck}}\right] - [(35.82K_a\gamma_{bf} + 6.26)\phi_G + \{942.38K_a + 186.77 + (0.025Q_k + 0.32q_k)\}\phi_Q] * 10^6$	
16	Failure of Abutment Wall in Shear.
$G(X)_{16} = 0.276f_{ck}(1 - 0.004f_{ck})bd\phi_R - [21.07K_a\gamma_{bf}\phi_G + (219.56K_a + 49.15)\phi_Q] * 10^3$	
17	Failure of Abutment Base in Bending.
$G(X)_{17} = \phi_R A_s(t) f_{yk}(t)\left[d - \frac{A_s(t) f_{yk}(t)}{2bf_{ck}}\right] - [(251.86 + 1.26\gamma_{bf})\phi_G + (0.08Q_k + 0.98q_k)\phi_Q] * 10^6$	
18	Failure of Abutment Base in Shear.
$G(X)_{18} = 0.138f_{ck}(1 - 0.004f_{ck})\phi_R bd - [(1076.42\phi_G + 5.40\gamma_{bf}) + (0.34Q_k + 4.22q_k)\phi_Q] * 10^3$	
19	Failure of Abutment Base Pile Group.
$G(X)_{19} = 9[(L_b + W_b)H_b C_s + C_b L_b W_b]\phi_R - [(3{,}238.28 + 16.20\gamma_{bf})\phi_G + (1.01Q_k + 12.66q_k)\phi_Q]$	

The time variant reliability indices for the components of the reinforced concrete bridge at a given point-in-time during its life-cycle were determined using a developed component reliability analysis sub-program in-built into the main program BRELA (Bridge RELiability Analysis). The increase in live load (truck load growth) and the deterioration of the structure over time due to chloride induced corrosion was modelled into the limit state functions derived for the bridge components. Load growth factors and corrosion rates which bring about changes in the performance variables (resistance and load) of the structure at any point-in-time were used in arriving at the components reliability indices. The First Order Reliability Method (FORM) was used to compute the components reliability indices with respect to the occurrence of each possible failure mode for the bridge components.

In order to compute the system reliability of the bridge over time, a system reliability analysis sub-program in-built in the main program BRELA was used. The results of the component reliability (safety indices of the components) at any point-in-time are imputed in the sub-program which was developed using the Ditlevsen's bounding method.

5. SERVICE LIFE PREDICTION AND OPTIMIZATION OF PREVENTIVE MAINTENANCE STRATEGIES USING THE GENETIC ALGORITHM APPROACH

The time variant reliability indices of the system were used in predicting the performance of the bridge at a given point-in-time over its service life. A target system reliability index β_{sys}^T of 3.0 (acceptable performance level) adopted in this study was used to predict the time during the service life of the bridge when this performance level becomes unacceptable. In order to ensure that the acceptable performance level is not violated, preventive maintenance measures were proposed on the components to militate against the loss of performance.

A sub-program for maintenance optimization in-built in the main program BRELA was developed for both the service life prediction and preventive maintenance optimization. Running the program requires inputting a target system reliability index, corrosion rates and a discount rate. The reliability of the system at a given point-in-time was computed and compared with the given target value, if the target performance level is not violated the program selects the 'do nothing' option otherwise other preventive maintenance options are chosen. The essence of the preventive maintenance measures is to mitigate the effect of corrosion on the reinforcement (which reduces the structural capacity of the structure) by either preventing chloride ingress or stopping corrosion propagation beyond the critical level. The preventive maintenance measures used in this study are defined as options (Chromosome) and each gene (parameter) represents an action within the strategy. These actions with the exceptions of 'do nothing' are selected every five (5) years which is assumed to be the period of inspection/validation of the bridge condition (Tantele, and Onoufriou, 2006).

Table 3 shows the preventive maintenance measures used in this study with their genetic codes, effective times and costs of application. The costs were adopted from existing literature (Tantele *et al.*, 2014) and converted to Naira using the prevailing exchange rate as at the 14th of March, 2017.

Table 3: Service Life and Cost of Preventive Maintenance Measures.

Preventive maintenance measure (I)	Genetic Code (II)	Effective time (years)(III)	Cost (N/m2) (IV)
Do nothing	1	N/A	N/A
Silane	2	5	1,668.95
Polyurethane Sealer	3	5	2,169.64
PM coating	4	10	12,684.02

This study adopted a multi-objective framework to optimize the lifetime preventive maintenance of a reinforced concrete bridge structure using point-in-time performance indicators. The performance indicators are the reliability indices associated with each component (deck, exterior and interior beams, pier cap, pier, pier base, abutment and abutment base) of the bridge as evaluated by the First Order Reliability Method (FORM) using the MATLAB-based program developed and the overall system safety index.

6. PROGRAM DESCRIPTIONS

6.1. Component Reliability Analysis Program

The program flow chart for MATLAB implementation of FORM for component reliability analysis is shown in Figure 2

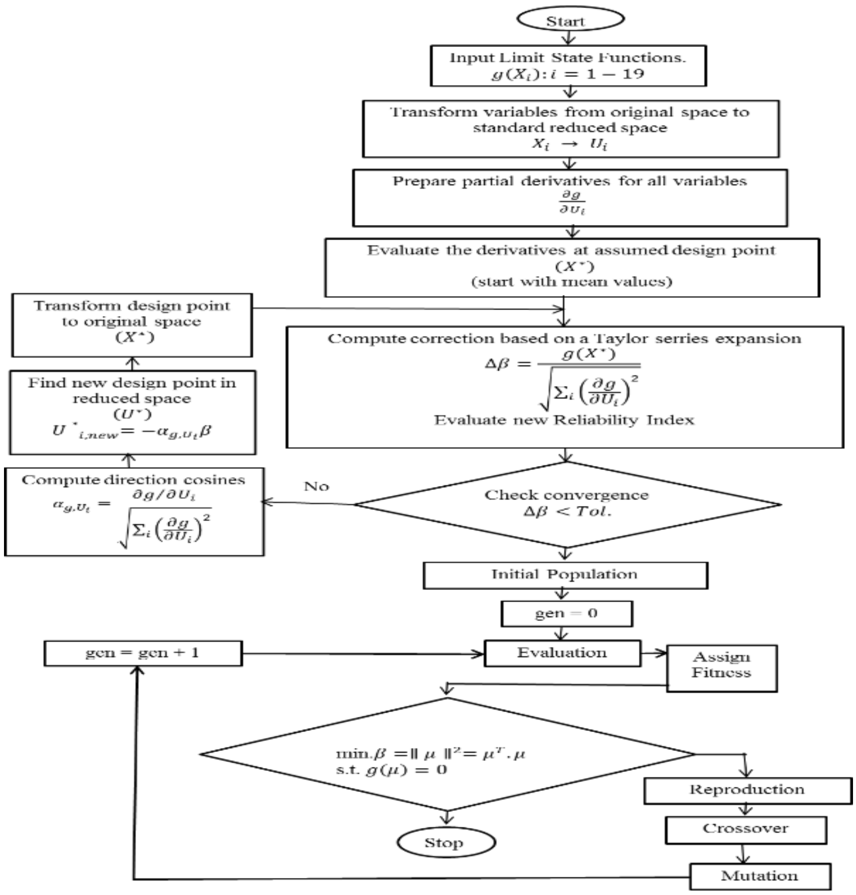

Figure 2: Flow Chart for MATLAB Implementation of FORM for Component Reliability Analysis

6.2. System Reliability Analysis Program:

System reliability analysis program is a MATLAB (MATLAB, 2013) coded computer program inbuilt into the main program BRELA and developed to calculate the system reliability using the bounding method. The system reliability is computed at different time intervals over the bridge service life by entering the component safety indices for a particular period under consideration. The program flow chart is shown in Figure 4.

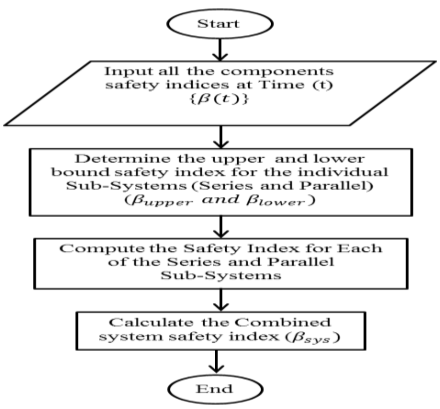

Figure 4: Flow Chart of the System Reliability Analysis Program

6.3. Maintenance Optimization Program:

This program also coded in MATLAB (MATLAB, 2013) and is in built into the main program BRELA, was developed to optimize the preventive maintenance strategies with respect to total life cycle cost using the genetic algorithm (GA) approach. The program was designed to calculate the system reliability of the bridge at a particular point in time and compare it with the given target. At any given period if the calculated value was above the target value it selects the "do nothing" option implying that no maintenance action was required until the next maintenance period. Otherwise, the program using all feasible combinations of the preventive maintenance options (Silane, Polyurethane sealer and PM coating) and the expected service life of the structure, optimize the preventive maintenance strategy by minimizing the total lifetime maintenance cost while maintaining the prescribed level of reliability. The total maintenance cost is obtained by first summing the cost of maintenance for each component over the service life and then that for all the components. The flowchart for the entire process is shown in Figure 5.

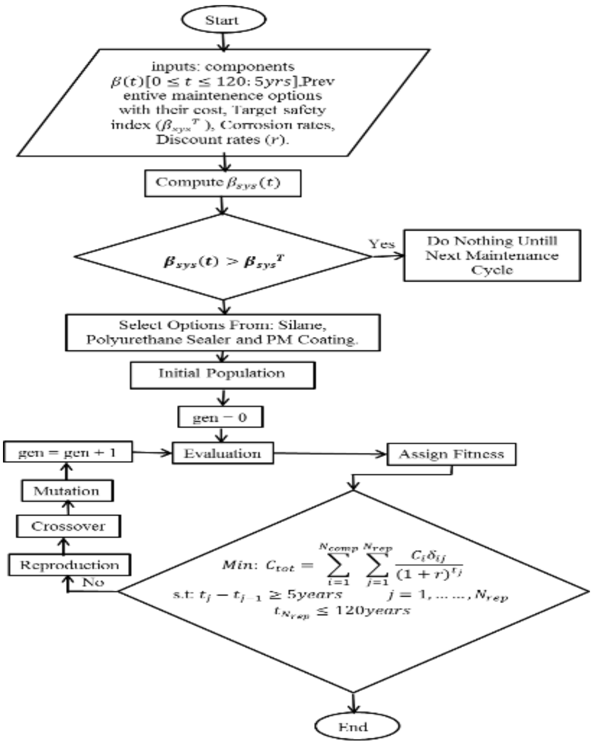

Figure 5: Flow Chart of the Maintenance Optimization Program

7. RESULTS AND DISCUSSION

7.1. Components and System Reliability Analysis.

The time variant reliability index represents the safety level of a system or its components at a given point-in-time during its life-cycle. The time-variation of reliability indices for the components and the overall system under the effects of both corrosion and live load growth are presented in Figures 6 to 12. The computational procedure has to be repeated every ten years over an exposure and service life of 120 years by considering the resistance and load variation over time, due to structural deterioration phenomenon (corrosion of steel reinforcement) or increasing demand (load growth).

Figure 6 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 most of the structural failure modes meet the target reliability index value of 3.8 prescribed for reinforced concrete bridges in the EN 1990 (2002) for a reference period of 100 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. 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. The effect of load growth on the component and system safety indices is shown in Figure 7; it has been assumed that the structure is not undergoing corrosion. The rate of capacity loss is low for most of the components failure modes and the overall system, except for failure mode3 which is failure of interior beam in bending. Overall the cumulative capacity loss is 34.47% for the most vulnerable failure mode to 0.00% for the safest failure mode (failure of abutment due to overturning) after 120years exposure and service. The system safety index decreased from an initial value of 3.08 at bridge age of 0 years to 2.29 at bridge age of 120 years (Figure 7) indicating a system performance loss of 25.65%.

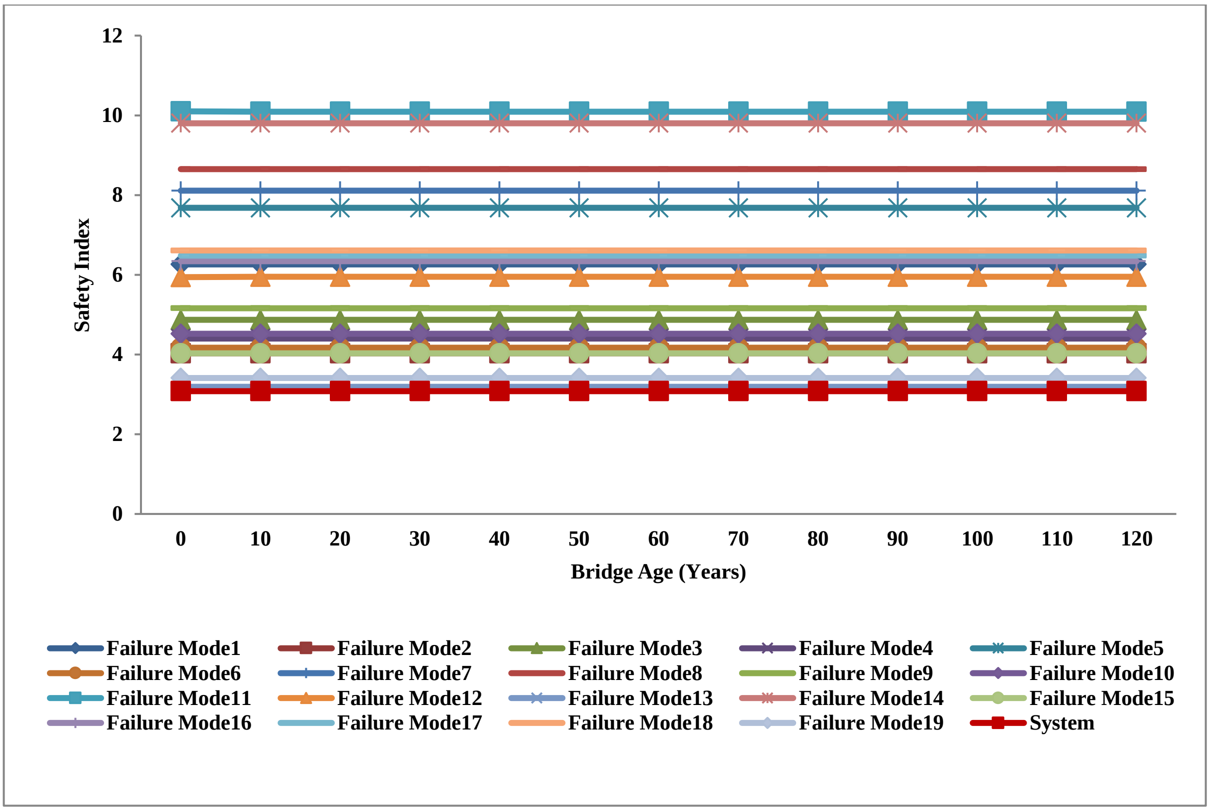

Figure 6: Variation of component and system safety index against bridge age for no corrosion without load growth

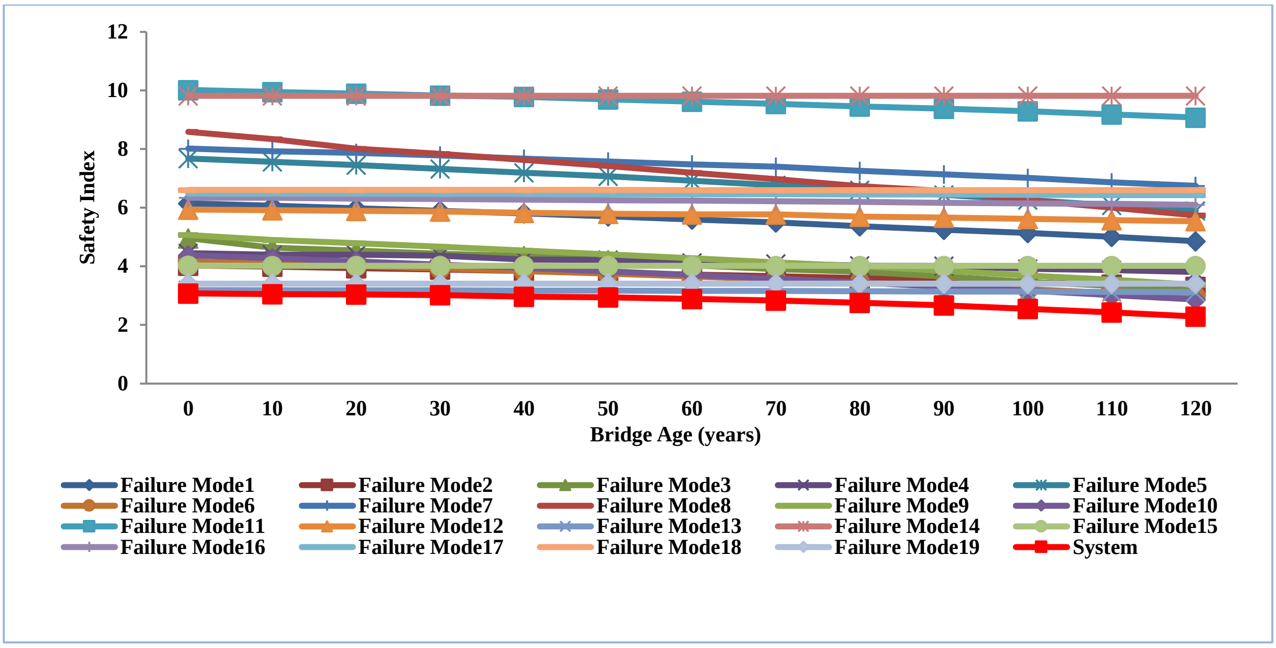

Figure 7: Variation of component/system safety index to bridge age for no corrosion, at load growth rate of 0.005

Figure 8 shows the system reliability and the reliability of the nineteen failure modes for the bridge components at a corrosion rate of 0.02 mm/year and truck load growth rate of 0.005. it can be seen that the reliability indices start to decrease after the initiation of corrosion (set here as 15years) over time. After 120 years of exposure and service, the cumulative reliability index decreased by 0.00% for the safest failure modes (failure modes 14 and 19) to 58.45% for the most vulnerable failure mode (failure mode10) and the system safety index of the bridge decreased by 51.30%. It is worth observing that in the early life of the bridge, the pier base pile group and abutment base pile group failure modes (that is failure modes 13 and 19) had the lowest initial component reliability indices but with negligible deterioration rate. As the exposure and service life increase there is a cross over point around 70 years for failure mode 10 and 90 years for failure mode 9 (which are the pier failure modes). Beyond these points the reliability of these failure modes becomes less than that of Failure mode 13 and failure mode 19. Taking decisions based on the initial reliability indices of components regarded to be safe in the early life of the structure may be misleading. This is because the varying rates of deterioration in the safety indices due to exposure conditions will make components initially termed safe to become unsafe during the later age of the structure.

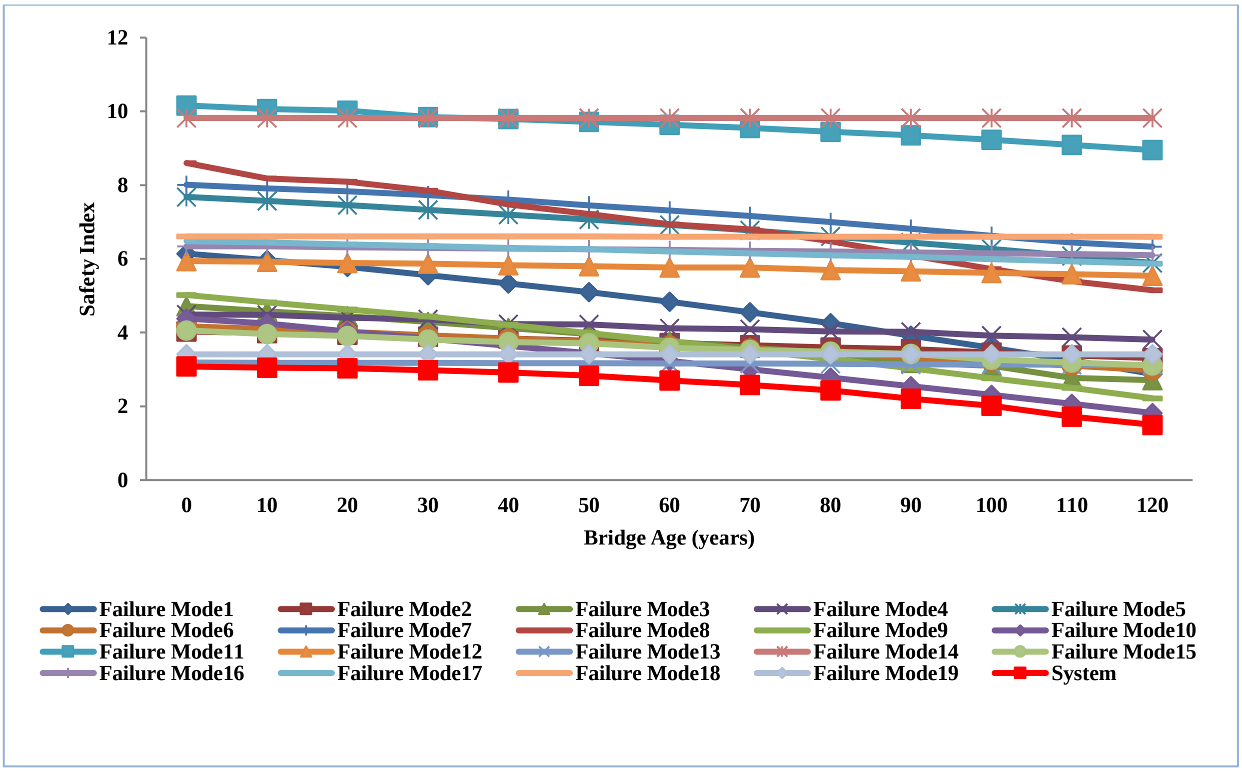

Figure 8: Variation of component/ system safety index to bridge age for corrosion rate of 0.02 mm/year and load growth rate of 0.005.

The effect of increasing the corrosion rate to 0.04 and 0.06 mm/year were respectively implemented and under these scenarios, Failure mode 1 experiences the highest capacity loss, with its safety index decreasing by 99.88% (after 120 years) and 100% (after 90 years) respectively. It was closely followed by failure mode 10 with the structural capacity decreasing by 85.84% (after 120 years) and 100% (after 110years) respectively, then failure mode 9 decreasing by 81.47% (after 120 years) and 100% (after 120years) respectively. The system reliability also decreased faster under these conditions with performance losses of 100% each after 120 years and 90 years of exposure respectively. This indicates that Failure modes 1, 9 and 10 have significant influence on the performance of the bridge as a system, thus the performance of the system can be assured by improving the reliability of these components.

In order to observe the effect of only corrosion and live load growth on time-dependent performance of the bridge, the system reliability curves were split. This was performed by first computing the reliability indices under time dependent corrosion and live load. Secondly, by keeping the live load constant at the initial value and computing the reliability indices under time-dependent corrosion only.

Figure 9 shows the relationship between system safety index and bridge age at varying corrosion rates of 0.00, 0.02, 0.04 and 0.06 mm/year respectively at a truck load growth rate of 0.005. It can be observed that the safety index was initially almost flat at about 3.08 for a period of 20 years; dropped gradually and then sharply as the corrosion rate increase with exposure. The higher the corrosion rate the steeper is the curve towards zero indicating a faster performance loss.

The scenario in Figure 10 gives a situation where there is no truck load growth. The curve for zero corrosion is flat at a safety index value of 3.08 throughout the exposure period of 120 years, however as the corrosion rate increases the curve deviates from the horizontal dropping downwards. For corrosion rate of 0.02 mm/year the drop starts after an exposure period of 60 years, the corresponding periods for corrosion rates of 0.04 and 0.06 mm/year were 30 years and 20 years. This further affirms the fact that the performance loss is greatly influenced by the rate of corrosion of the steel reinforcement. Adequate provisions to stem this effect need to be put in place so as to safeguard a structure when exposed to an aggressive environment.

Figure 11 compares the system reliability with those of the failure modes in bending for the components subjected to corrosion and load growth. A wide gap is observed at the beginning of the exposure period between the system safety index curve and the curves of the safety indices for the various component failure modes. These however converge towards the system safety index as the exposure period increases, indicating the high rate of deterioration due to the combined effect of corrosion and load growth. The reverse is the case as can be observed from the plot of the failure modes in Figure 12, for members in shear, overturning and bearing failure (failure of pile group) which are affected only by truck load growth. At the beginning of exposure, the system safety index is close to those of the components but diverges as the exposure period increases, indicating the slower rate of deterioration and loss of carrying capacity of the structure since these failure modes are affected by only load growth.

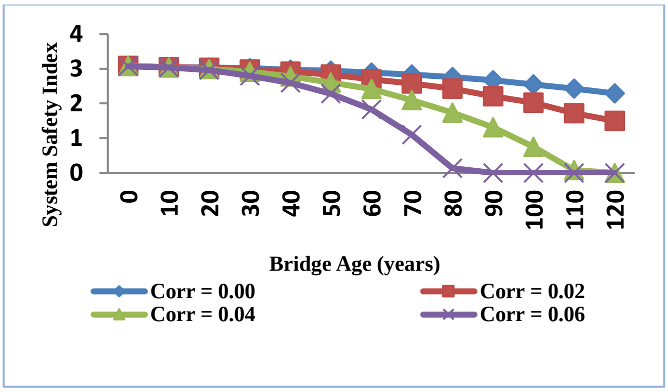

Figure 9 : Variation of system safety index to bridge age at varying corrosion rates for load growth rate of 0.005.

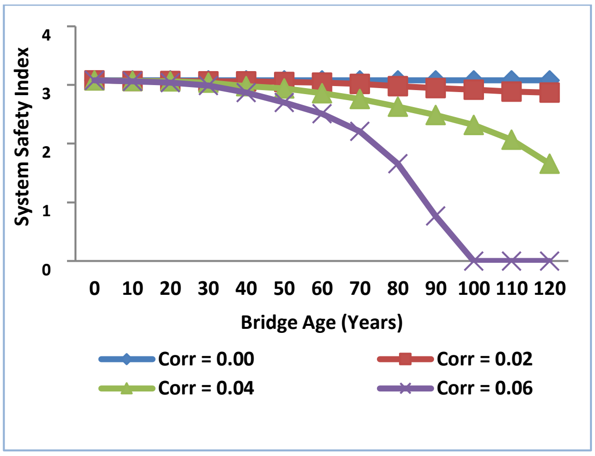

Figure 10: Variation of system safety index to bridge age at varying corrosion rates with no load growth

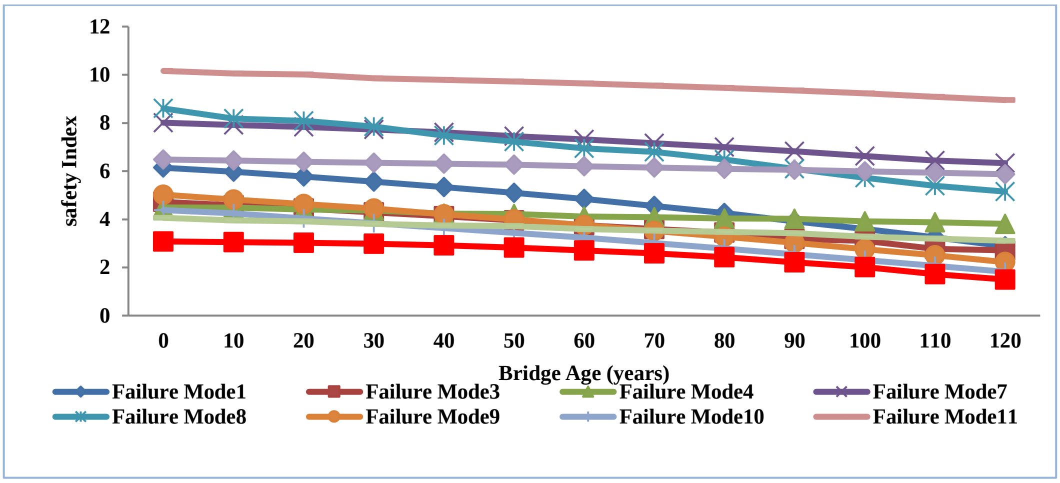

Figure 11: Variation of safety index of system and failure modes affected by both corrosion and load growth against bridge age for corrosion rate of 0.02 mm/year and load growth rate of 0.005

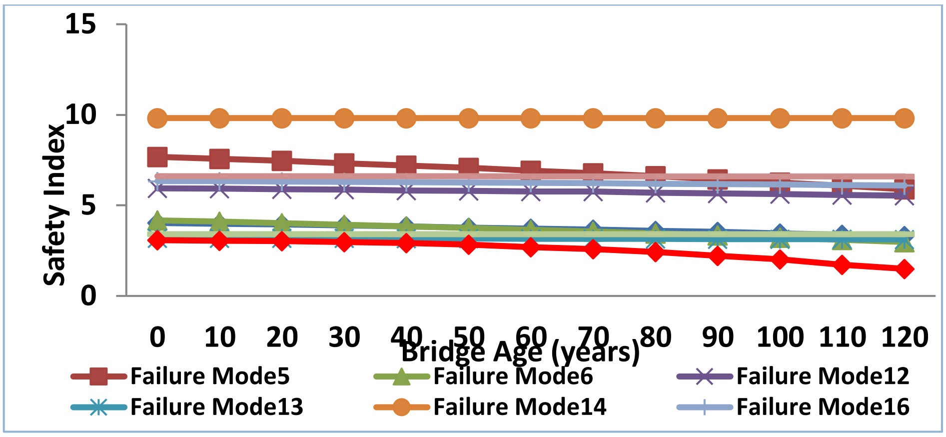

Figure 12: Variation of safety indices of system and failure modes affected by load growth only against bridge age

7.2. Optimum Preventive Maintenance Strategies

Optimum preventive maintenance strategies were developed by using the preventive maintenance optimization sub-program in BRELA which selects all feasible combinations of the various options listed in Table 3 for each components of the bridge. The preventive maintenance strategies for the various bridge components at corrosion rates of $0.02\ mm/year$ which are the optimized combination as generated using the genetic algorithm in BRELA is presented in Tables 4 for a target system reliability index, $\beta_{sys}^{T} = 3.0$.

Table 4: Preventive Maintenance Strategies for the Bridge Components at corrosion rate =$0.02mm/year$*, discount rate =*3%*,* $\beta_{sys}^{T} = 3.0$.

COMPONENT	LIFE TIME PREVENTIVE MAINTENANCE PLANS (YEARS)																								
	0	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100	105	110	115	120
Deck	1	1	1	1	1	1	2	3	3	2	2	4	1	4	1	3	3	2	3	4	3	3	2	2	2
Interior beam	1	1	1	1	1	1	1	3	4	1	3	3	2	2	3	4	1	4	3	2	3	4	1	2	2
Exterior beam	1	1	1	1	1	1	3	2	3	4	1	4	1	2	2	2	2	2	2	2	2	2	3	4	1
Pier cap	1	1	1	1	1	1	2	4	1	2	2	3	4	1	2	3	3	3	4	1	4	1	3	2	2
Pier	1	1	1	1	1	1	2	4	1	2	2	2	2	2	3	2	3	2	3	4	1	3	3	2	2
Pier base	1	1	1	1	1	1	4	1	4	1	3	4	1	2	2	3	4	1	2	2	4	1	3	2	2
Abutment wall	1	1	1	1	1	1	1	2	2	3	3	4	1	3	3	4	1	4	1	3	2	2	3	2	2
Abutment base	1	1	1	1	1	1	3	2	4	1	2	2	2	3	2	2	4	1	4	1	3	2	2	2	3

Decoded action: 1-Do nothing, 2-Silane, 3-Polyurethane sealer, 4-PM coating.

Total life-time preventive maintenance cost = ₦ 86,251,336.00. Total discounted life-time preventive maintenance cost as a function of time = ₦ 2,484,899.66. Using a target system reliability index $\beta_{sys}^{T} = 3.0$ and corrosion rate of $0.02mm/year$ the structure tends to be at high risk of failure around year 35, Hence , preventive measures must be put in place before it reaches this age (implemented here at year 30). Table 4 shows the matrix of the preventive maintenance options for the various components of the bridge required to ensure that the reliability of the bridge system remains above the target value when the structure is undergoing corrosion at the rate of 0.02mm/year. The "do nothing" option (coded-1) is selected at the beginning of the strategy (0-25 years) for all the components since the reliability of the system is above the target value. As shown in Table 4 various combination of "do nothing", silane (coded-2), polyurethane sealer (coded-3) and PM coating (coded-4) are selected in the subsequent intervening periods in such a way that the overall whole life cycle cost is minimized. The available options; silane, polyurethane sealer each have an effective period of five years while PM coating has an effective period of ten years. Thus, each time PM coating is selected by the program at a given period it is followed by the "do nothing" option in the subsequent period so as to take care of the ten-year effective time. The optimum total life-time preventive maintenance cost for this strategy is ₦ 86,251,336.00 and the discounted cost as a function of time is ₦ 2,484,899.66. Similar optimized combinations using the developed sub program using genetic algorithm was implemented for corrosion rates of 0.04 mm/year and 0.06 mm/year, and the optimum total life-time preventive maintenance cost for the strategies were observed to be ₦ 104,215,913.80 and ₦ 119,820,596.30 with the discount cost as a function of time to be ₦ 3,002,441.05 and ₦ 3,452,056.18. respectively. The total life-time preventive maintenance cost increased from ₦ 86,251,336.00 - ₦ 119,820,596.30 as the corrosion rate increased from 0.02-0.06 mm/year.

8. CONCLUSION

The following conclusions were drawn from the study:

i. The reliability of the components before and after the onset of corrosion computed over the service life of the reinforced concrete bridge (120 years), shows a capacity loss range 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.

ii. The reliability of the system decreases over time due to the effect of the deteriorating components on the bridge system. Components with an initial low reliability level do not necessarily control the reliability of the system. This is because in the early life of the bridge, components whose reliability level is dominated by live load increase will have a significant effect on the system's performance. However, in the later stage the components whose performances are greatly affected by corrosion become more significant. Therefore, the most important component in the early life of the system may not be the most important during the later period.

iii. The system reliability of the bridge over time shows a system performance loss of 25.65% when there is no corrosion, 51.30% for a corrosion rate of 0.02 mm/year, 100% (at 120 years) for a corrosion rate of 0.04 mm/year and 100% (at 90 years) for a corrosion rate of 0.06 mm/year when the truck growth rate is 0.005. Lower loss of performance was observed where there was no growth in traffic load. System performance losses of 0.00%, 6.82%, 46.10% and 100% (at 100 years) were obtained when the corrosion rate was varied from 0.00, 0.02, 0.04 and 0.06mm/year respectively. The results of the system reliability show a decrease in the system safety indices as the exposure time increases, the rate of decrease or deterioration however depends on the exposure condition.

iv. The violation of a given target system reliability index (performance level), is dependent on the rate of structural deterioration. For a target system reliability index of 3.0 and load growth of 0.005, preventive measures have to be implemented around year 30, 25 and 20 for corrosion rates of 0.02, 0.04 and 0.06 mm/year respectively.

9. NOTATIONS

$\emptyset_R$ Resistance Model Uncertainty
$\emptyset_G$ Permanent Load Model Uncertainty
$\emptyset_Q$ Traffic Load Model Uncertainty
$Q_k(t)$ Truck Traffic Load
q_k UDL traffic load
f_{ck} Concrete compressive strength
$f_{yk}(t)$ Steel strength at time t
$A_s(t)$ Area of steel at time t
A_p Cross sectional area of pier
b Width of beam, pier cap, Pile cap, pier base, abutment wall
d Effective depth of beam, pier cap, abutment wall
L_b Length of pile block
W_b Width of pile block
H_b Height of pile block
C_s Ave. cohesion around pile group
C_b Ave. cohesion beneath pile group
$D(t)$ Diameter of reinforcement bar at time t:
γ_{bf} Unit weight of backfill material
K_a Active pressure coefficient
h_w Ground Water level

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