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Toward Preventive Maintenance and Rehabilitation of Bridges in Cameroon

By

Lezin SEBA MINSILI[1*], Michel MBESSA[2],
Crispin PETTANG[3], Nadia DAMANDI TARANG[4]

ABSTRACT :
Most bridges in Cameroon built in postcolonial period are more than fifty years old and are in dilapidated structural state. This situation was caused by the inadequacy of trained technical experts and of required maintenance programs that were tributary to the scarcity of limited financial means. Since a bridge has a prescribed lifespan with decreasing structural capacities, we must guarantee its safe function using appropriate inspection and monitoring methods. This paper outlines some aspects of research and development related to ongoing bridge curative and preventive maintenance in the country. The proposed methodology expresses the bridge reliability index as a functional value of its bearing capacity. Results, obtained through application of the methodology on local bridges, show that it is possible to extend the bridge lifespan for more than thirty years if appropriate curative and predictive maintenance works are done in due time. Technical officials in the Country presently are seeking means to implement these findings to the national bridges network.

Keywords: bridge maintenance, monitoring, reliability index, corrosion, fatigue, repair.

1. Introduction

The vast majority of bridges in Cameroon were built more than 50 years ago and 54% of the country bridges network is relatively in a bad state [1]. With the high demand of heavy axle loads in the country to carry goods from the inland to sea ports and vice-versa, many bridges are subjected to loads far higher than those envisaged during the design stage. The poor state of these bridges is worsened with the insufficiency of maintenance investment so that many of existing bridges have significantly deteriorated over their years of service as seen in Figure 1 with the collapse of the former Mungo bridge in July 2004.

| [1]IABSE Member, Senior Structural Engineer, Ph.D., Senior Lecturer, LMM Laboratory, ENSP. The University of Yaoundé I, P.O. Box 8390 Yaoundé, Cameroon
[2] Senior Civil Engineer, Ph.D., Senior Lecturer, LMM Laboratory, ENSP. The University of Yaoundé I
[3] General Civil Engineer, Ph.D., Professor, LMM Laboratory, ENSP. The University of Yaoundé I
[4] Civil Engineer, Graduate Student, LMM Laboratory, ENSP. The University of Yaoundé I
* Corresponding Author.

An early diagnostic of bridge damage is always necessary since it reduces maintenance cost and increases the structural safety and reliability. While visual inspection fails to assess the damage at early stages in order to apply recommended maintenance measures, vibration measurements are sufficiently sensitive to detect damage even when it is situated in hidden or internal areas. Therefore, there is a growing interest among local highway and railways authorities in supporting related research programs for the mastering and development of vibration-based damage detection methods as well as progressive maintenance methodologies.

Many studies have focused on preventive and corrective maintenance techniques using different models and constraints to optimize the decision making. With a given bridge network Li et al. [2] and Farid et al. [3] developed a web-based prototype introducing a sorted list of bridges which gives higher priority ratings to bridges with greater needs for maintenance and rehabilitation with details on the specific repair method, cost, and expected improvement. Considering damage detection using mode shapes and frequencies, Carden and Fanning [4] underline the difference between experimental and numerical modes with the help of the displacements difference and of objective functional of strain energy, of modal flexibility, and of residual forces [5, 6] to locate and quantify an experimental damage. Damage is detected through the comparison between the undamaged and damage state and it is modeled as a reduction of stiffness (or of the bearing capacity).

This paper outlines some aspects of the research related to bridge monitoring and preventive maintenance in Cameroon with an application to the ongoing predictive maintenance work of the new bridge bridge over the Mayo-Boula River in the Far-north region. With the support and real monitoring data provided by LABOGENIE [7], a research team in the LMM laboratory at Yaoundé-1 University has made studies methodologies for condition assessment, monitoring-based strategies, sustainable damage detection, and of bridge preventive maintenance and management. Failure to perform a designed structural functionality might be associated to a deformed element, cracks, fatigue, corrosion, or a missing part, have been considered through a performance functional using the probabilistic theory of maintenance and the Frangopol's model associated with the reliability index. Obtained results display a bridge maintenance system that can be used to: (1) identify actual safety reserve of bridge; (2) predict the bridge response and corresponding deterioration; (3) forecast information for immediate safety assessment; and (4) provide outlines for planned and prioritized assessment and maintenance.

Figure 1:
(a)*The ollapsed former Mungo steel bridge and (b) the reconstructed new one.*

2. Materials and Methods

Basic principles of engineering structural mechanics are used to calculate the capacity of a bridge structure. In the present work, the bridge over the Mayo-Boula River (Figure 3) was declared unfit and unsafe for transportation and was destroyed after fifty years of loyal services without maintenance and rehabilitation works to improve its service ability and thus to extend its lifespan. The probabilistic modeling of the behavior the bridge under exploitation (or in the design stage) over the time history is critical in determining its remnant resistance and its capacity that can be improved if remedial measures are to be applied. An important component of bridge maintenance is concerned with the calculation or estimation of the probability of a limit state violation (occurrence of structural failure) for the bridge during its lifetime. Its estimate may be obtained using measurements of the long-term frequency of occurrence of the interested events on similar structures, or using numerical analysis and simulation of forecasted data. For example statistics of data for bridge elements are used in modeling bridge behavioral components such as strengths, sizes, deterioration rates, truck load magnitudes, traffic volume, etc.

Frangopol [10, 12] developed the basis for cost-effective bridge management incorporating lifetime reliability and life-cycle cost with realistic examples of optimum bridge maintenance planning based on minimum expected cost. The reliability index (β) is used as a measure of a bridge safety. In general, individual bridges in a bridge group have different ages and their reliability is time dependent. The service life of an individual bridge of a bridge network is a time dependent progression of its elements reliability states (numerically from an excellent state to unacceptable state). The random variables affecting lifetime reliability of a bridge include the initial reliability index, the time of damage initiation, the reliability deterioration rate without (or with) maintenance. It is therefore possible to graph the behavior of the given random variable using its most important characteristic parameters such as the mean value (μ) and the standard deviation(σ), and a dimensionless measure of the variability that depicts the reliability index used to assess the importance of maintenance (or rehabilitation) measures to be implemented. The numerical expression of the time-dependent reliability index developed by Pablo [9] is given as:

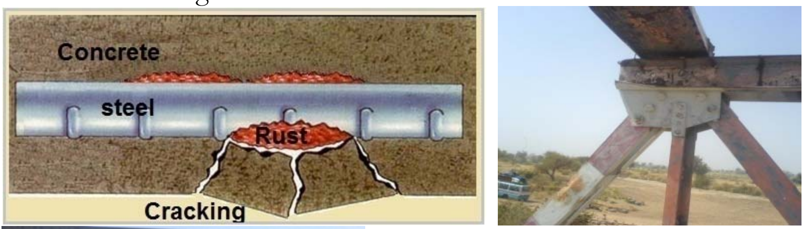
$$\beta(t) = \frac{\mu(g)}{\sigma(g)} \qquad (1)$$

where g is the function of the studied deterioration expressed as $R – S$; R is the resistance or the load-carrying capacity of the structural component, and S is the load effect or the load demand to the component, modeled as random variables because of the uncertainty associated with the material production, the preparation process, the construction quality control and the random load effect.

The given reliability index can be calculated using the First Order Reliability Method [10]. However, since both the load S and resistance R were assumed to be log-normally distributed, the calculation of the reliability index is simplified as:

$$\beta = \frac{\overline{\log \Delta\sigma_R} - \overline{\log \Delta\sigma_S}}{\sqrt{S^2_{\log \Delta\sigma_R} + S^2_{\log \Delta\sigma_S}}} \qquad (2)$$

The calculation of each of these loads is characterized by the understanding of the considered degradation functional. Report obtained in previous inspections done by MINTP [8] shows that 90% of deteriorated civil engineering structures in Cameroon resulted from corrosion and fatigue of main structural elements such as steel bracings, crazing of the running surface and bridge scouring. The present work deals only with degradations related to corrosion and fatigue of structures in reinforced concrete and steel as seen in Figure 2.



a) Embedded steel corrosion in concrete;
b) steel element corrosion;
c) steel element failure

Figure 2: *Degradations on concrete and steel structural elements due to corrosion [8]*

2.1 Remnant Fatigue Related Capacity

Establishing the remnant bearing capacity of a bridge results in estimating the structural deterioration in an existing steel/concrete bridge and, at the same time, in determining the remaining structural resistance to loading in its lifespan according to the Linear Elastic Fracture Mechanics theory by means of detailed loading history analysis (Alessio Pipinato, 2012, [11]). The probability of failure can be expressed with the reliability index according to the normal standard distribution as seen in Equation 1, and the reliability of a structural element is compared to the target value:

$$\beta_{\text{fail}} \geq \beta_{\text{target}} \qquad (3)$$

where β_{fail} is the reliability index with respect to failure; β_{target} is target reliability index.

This model implies the use of the fatigue action effect (the required nominal fatigue strength) which is obtained by dividing the required nominal fatigue strength by the action effect of the fatigue load:

To evaluate the fatigue effect in a steel structural element, in-situ experimental data are required to obtain appropriate Wöhler curves, known as characteristic S-N curve approach. Taking advantages of existing normative documents on various experimental settings and on known information of a given bridge, the relationship between the number of stress cycles (***N***) and the stress intensity range (**Δσ**) inside the considered structural element, is given as:

$$\boldsymbol{N = C\Delta\sigma^{-m}}$$

Where ***C*** and *m* are considered element material constants depending on the geometry function, the stress concentration and the crack propagation rate.

For a reinforced concrete bridge two types of failures are considered according to ZE Wu [27]: deteriorations due to bending stresses and to sheer stresses. Deteriorations due to bending are the result of over-compressed concrete surface and of over-tensed steel reinforcement leading to their corresponding ultimate limit state, while deteriorations due to shear forces are the result of over-reinforced concrete beams when micro-cracks from bending intercept those from shear deformation according BAEL 91 [12]. The following expression gives the relation between the stress cycle number (*N*), the maximum (σ_{max}) and the minimum (σ_{min}) stress intensities that depicts the fatigue effect in a considered concrete bridge :

$$N = N_o \times \left(\frac{\delta\sigma_o}{\sigma_{max}-\sigma_{min}}\right)^k \text{for} \sigma_{max} - \sigma_{min} \geq \delta\sigma_o$$
$$N = N_o \times \left(\frac{\delta\sigma_o}{\sigma_{max}-\sigma_{min}}\right)^{k'} \text{for} \sigma_{max} - \sigma_{min} \leq \delta\sigma_o \quad (5)$$

where $\sigma_o=0.3\times f_e$; *fe* is the concrete elastic limit; $k=9$; $k'=k+2=11$; $N_o=10^7=$ the estimated number of cycle at failure.

2.2 Remnant Corrosion Related Capacity

Concrete is alkaline due to the presence of hydroxides of calcium, potassium, and sodium ($Ca(OH)_2$, *KOH*, and *NaOH*) and the alkalinity typically ranges from *PH* 12 to 13 according to Shruti Sharma [13]. Due to the high alkalinity of the concrete pore water, the steel reinforcing bars are passivated by an iron oxide film (Fe_2O_3) that protects the steel. Corrosion is basically an electrochemical process where the anode and the cathode are on the same steel bar. At the anode, iron atoms lose electrons to become iron ions (Fe^{++}). At the cathode, oxygen in the presence of water accepts electrons to form hydroxyl ions (OH^-). Concrete works as an electrolyte that facilitates the flow of electrons between the anode and the cathode. Both the anodic and cathodic reactions are necessary for the corrosion to occur and they need to take place simultaneously. The hydroxyl ions combine with the ferrous ions to form ferrous hydroxide $Fe(OH)_2$. In the presence of water and oxygen, the ferrous hydroxide is further oxidized to form Fe_2O_3.

Chloride ions are introduced into the concrete by marine spray, industrial brine, and chemical treatments. These chloride ions can reach the reinforcing steel by diffusing through the concrete or by penetrating cracks in the concrete. Corrosion of steel embedded in concrete, in the presence of chlorides but with no oxygen (at the anode), takes place in several steps. At the anode, iron reacts with chloride ions to form an intermediate soluble iron-chloride complex:

Anodic reaction :$Fe+2Cl^- \rightarrow (Fe^{2+}+ 2Cl^-)+2^{e-}$

Cathodic reaction :$(Fe^{2+}+ 2Cl^-)+2^{e-}+2H_2O \rightarrow Fe(OH)_2+2H^+2Cl^-$ (6)

When the iron-chloride complex diffuses away from the bar to an area with higher PH and concentration of oxygen, it reacts with hydroxyl ions to form $Fe(OH)_2$. This complex reacts with water to form ferrous hydroxide as seen in Equation 6. The ferrous hydroxide is oxidized to rust in the presence of oxygen and water. The manifestation of corrosion is in the formation of these corrosion products on the surface of steel reinforcement. Thus, the steel–concrete interface is altered where rust is formed. Due to higher volume of rust than the corresponding mass coating of steel, an outward pressure is generated. As concrete is weak in tension, cracks are developed in it leading to debond as seen in figure 2.a.

For steel structures, it is difficult to quantify the level of degradation and damages caused by corrosion. But there are a multitude of anti-corrosive products (galvanization, painting and cathodic protections) in the market with corresponding useful technical guides, what make it easier to define and to predict specific functional behaviors.
Environmental conditions (chloride ions in marine and industrial milieus, carbonation due to atmospheric Carbon Dioxyde (CO_2), and rain water) are the main factor causing corrosion of bridges in Cameroon. The state of corrosion and its action on a structural element/system can be characterized by the degree of carbonation (X_C) in the depth from the given element surface, and is given by the following expression:

$$X_C = \gamma \times f(HR) \times k \times \sqrt{t}$$

with the structure being exposed to rain, γ=0.9; *k* is the transportation coefficient of concrete given as $\sqrt{365} \times (\frac{1}{2{,}1\times\sqrt{f_{C28}}} - 0{,}06)$; the influence of relative humidity (*HR*) is expressed as $f(H;R)=-3.5833\times HR^2+3.483\times HR+0.2$.

3. Results and Discussions

a)

b)

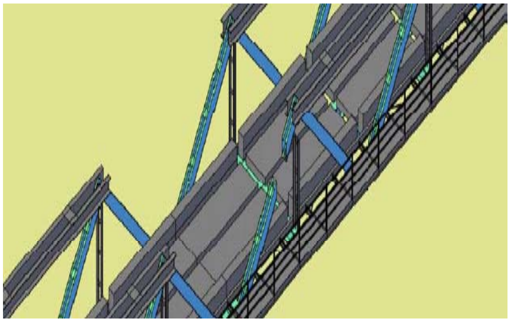


Figure 3: the former bridge over the Mayo-Boula River (a), and its numerical failure model (b)

The superstructure of the former bridge over the Mayo-Boula River was a two-span steel Warren truss supporting a one-line highway concrete board, 4.2 meters wide with two sidewalks, resting on transverse beams connecting the two truss systems of the bridge. Steel truss elements where IPE350 and grade S235 from the EUROCODE 3 standard. The total span of the bridge was 50 m and consisted of two continuous spans of 25 m each, resting on a middle pier and two abutments. The bridge was constructed on the colonial era before the year 1960, and according to residents of Mayo-Boula, the bridge lasted 50 years before its decommissioning in 2009.

In the absence of engineering early data recording on the bridge management from local highway authorities, the bridge auscultation campaign that took place in 1999 displayed that the bridge suffered from severe anomalies and damages ranking from material degradation to mechanical failure such as: corrosion of steel truss elements and concrete reinforcements, concrete cracking, concrete-steel debonding, truss elements failure, fatigue due to traffic load and breaking that can be seen in figures 2.c) and 3.b). The breakage of truss elements, due to a chock with an oversized trailer carrying a bulldozer in 2004, caused a 40 cm axial deformation of the bridge system as seen in figure 3.a) and forced highway authorities to reduce the running speed and circulation of heavy trucks on the bridge [13, 14, 18] for five years under high risk of failure.

3.1 Analysis of the former bridge

Using the developed system reliability procedure to assess the remnant exploitation lifespan of the initial steel bridge of the Mayo-Boula River, taking the reliability index as a functional of parameters developed in equations 2 and 5 for an individual steel element and for the whole bridge, the predicted lifespan is made known and have been fixed as n = 100 [9]. For each loading cycle, numerical values of maximal and minimal stresses of each element were computed using Finite Element software in structural engineering developed and tested in the Department of Civil Engineering of ENSP, The University of Yaoundé 1.

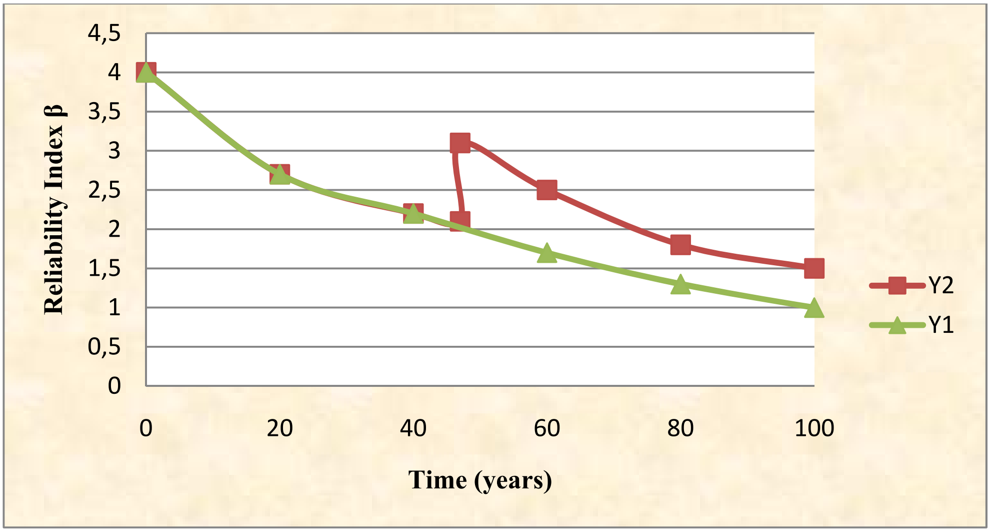


Figure 4: *Fatigue reliability index of the old bridge with (Y2) and without (Y1) punctual maintenance measures*

Various elements of the bridge system being classified into six groups of structural elements subjected to induced traffic loading, the Wöhler characteristic diagrams [14] is drawn for each elements group using results of stress analysis due to specified loadings , and thereafter the group performance or reliability diagram. From the performance diagram of each group we deduce the reliability index diagram of the bridge subjected to degradation due to fatigue of a given number of its constitutive elements described in figure 3.b) with computational results depicted in figure 4.

This graph shows that within a period of 100 years, the failure indices decreases from 4 to 1 (figure 4.Y1).The bridge being made up of redundant elements, the threshold security, or the allowable reliability index, that corresponds to the time when the bridge cannot withstand the designed traffic load is n=50 years or simply β=2. This implies that below this ultimate value one must implement adequate maintenance procedures to avoid an unexpected failure as the one depicted in figure 1.a). It will therefore be

judicious regularly to program bridge inspection and provide related curative maintenance to the failed bridge element or group of elements.
In this case by predicting the functionality, the bridge might have been maintained by replacing broken diagonal elements and deformed horizontal bottom chord elements in due time before the overall bridge was declared unfit to traffic. An operation that was able to extend the bridge life span for at least 30 years as it is shown in figure 4.Y2. Since it is not fit to wait for the bridge failure or the breakage of one of its element in order to apply curative maintenance, it is technically and economically convenient to apply preventive maintenance by forecasting the allowable reliability index for a particular element or group of elements that is subjected to specified traffic and environmental conditions.

3.2 Analysis of the new bridge

The new bridge (Figure 5) over the Mayo-Boula River is a three span reinforced concrete platform made of four precast simple supported beams spanning 17 m with a 0.80 m×1.50m cross-section each, and over which an in-situ 10-m wide board was casted, with 1.5 m sidewalk at each side. The running surface is a 8-cm thick bituminous concrete [**15**]. Specifications of the new bridges show that all deck concrete shall have a minimum compression stress of 25 Mpa and 2.1 MPa in tension considering that the cement mean stress was 35 MPa, with steel Reinforcement strength taken as fy=460 MPa and the cement concentration was 400 Kg/m^3.

Figure 5: *The reinforced concrete new bridge over the Mayo-Boula River*

Adopting the same principle and calculation procedure described for the former bridge by considering only the fatigue effect of steel reinforcement, the obtained diagram of the computed reliability index β is similar to figure 4.Y1 with β = 8.7 after construction and β = 5.4 after 100 years. These results show that the fatigue effect of steel reinforcement could be ignored in the maintenance process of the new bridge during its lifespan.
Beams undergoing accelerated environmental corrosion showed reddish brown patches of corrosive products and a longitudinal micro-crack that progressed along the direction of the reinforcement. With the increase in volume of corrosive products, more cracks also parallel to the steel reinforcement are observed after a relatively long period and a

reddish brown liquid oozed out of the reinforcement steel surface. This process, if not toppled by any curative rehabilitation, will increase with increase exposure of the reinforcement to surface environmental corrosion characterized by large and wide patches and cracks that debond the steel-concrete surface contact leading to a beam with highly dilapidated and poor structural conditions (Figure 2.a).

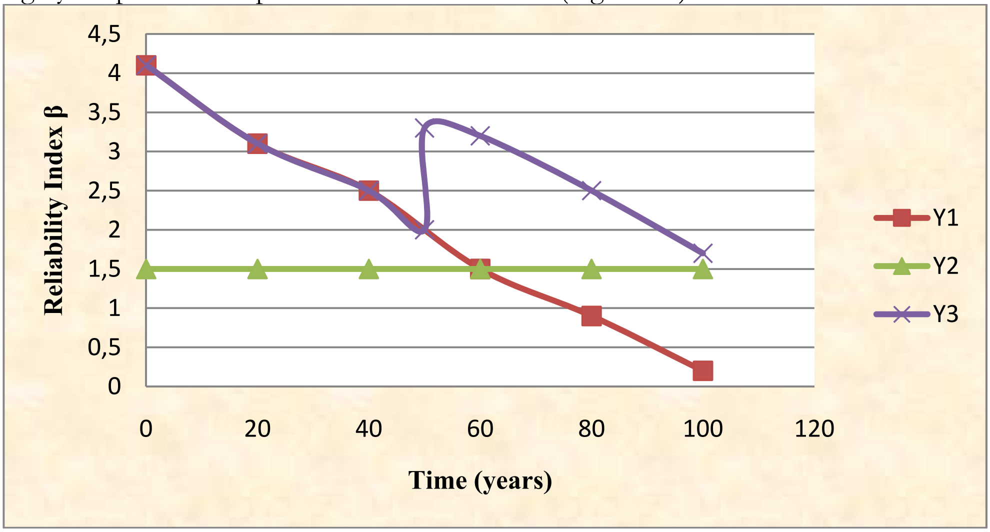


Figure 6: *Corrosion Reliability Index of the new bridge without (Y1) and with (Y3) punctual maintenance measures compared to the safety level (Y2).*

From data obtained on a local weather station and from execution methods of the new bridge, the reliability analysis is done with functional parameters developed in equations 2 and 7 for an individual beam and for the whole bridge. Analytical results of the reliability index history are presented in Figures 6. We notice that within a period of 100 years, the reliability index varies from 4 to 0.2 (figure 6.Y1). According to N. HYVERT [26], when the reliability index $\beta = 1.5$, the corrosion of the steel reinforcement is active and it is good to apply a curative maintenance process.

Once corrosion attacks a reinforced concrete structure, it is possible to remedy the situation by undertaking a repair and strengthening of the corroded area after 50 years. The present state of the art on rehabilitating existing concrete bridges subjected to corrosion process employs systems which have been developed through experience and which are empirical in nature. These include: replacing or adding reinforcing Steel bars that has been excessively corroded; repairing of minor spalls by shotcrete, epoxy-sand mortar, or non-shrink cementitious grouts; the use of an epoxy resin or polymer latex bonding coat to achieve a reliable bond; repair of existing cracks; patching with cementitious/resin mortars to upgrade weak honeycomb materials and voids; and concrete replacement with fiber reinforced concrete/mortar.

The diagram of bearing capacity of the bridge after the curative corrosion effect (figure 6.Y3) shows that the bridge can last more than 100 years if preventive maintenance procedures are applied in due time. The value of the observed upward jump in that diagram, after curative maintenance, depends on the cost intensity one puts in the process with the fact that many curative and preventive works must be accomplished during the bridge life-span in order to obtain corresponding refurbishing effect on the bridge capacity. If preventive maintenance procedures are not done during the beam lifespan, the value of the bearing capacity characterized by the reliability index down crosses the target value (figure 6.Y2, at n = 63 years), and the damage initiation starts as the bearing capacity deteriorates without restraint. Conversely, when preventive maintenance are taken in due time, the bearing capacity of the bridges abruptly increases leading to the extension of the bridge lifespan.

Conclusions

The safety of bridges is a key factor in promoting transport of goods, equipments and passengers required for the nation's economic growth. The changes in a bridge structural capacity can be adequately assessed using appropriate inspection and monitoring methods in order to guarantee its safe response to external loading and environmental effects. Being able to identify and predict a bridge structurally deficiency constitutes an important step in the hand of bridge engineers and managers to prioritize and to budget for actions to immediate/late repair, rehabilitation, or even replacement.

In this paper, a numerical approach to the reliability analysis of the former bridge on the Mayo-Boula River is presented, and the importance of applying appropriate curative and predictive maintenance measures in due time is underlined. The old and the new bridges are modeled using the Finite Element approach and the proposed methodology to express the reliability index of the bridge as a functional value of its bearing capacity. Obtained results on the effect of fatigue and corrosion are showing that, both for the former steel bridge and the new reinforced concrete bridge, it is possible to extend the bridge lifespan for more than thirty years if appropriate curative and predictive maintenance works are done in due time.

Present results have fostered the LABOGENIE and the LMM laboratory to officially draw the importance of implementing reliability based curative and predictive maintenance procedures in the existing and future bridges network of the country. Procedures presented to national regulatory and construction agencies included measures to: (1) identify actual safety reserve of bridge; (2) predict the bridge response and corresponding deterioration; (3) forecast information for immediate safety assessment; and (4) provide outlines for planned and prioritized assessment and maintenance.

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