Long Term Evaluation of the Structural Reliability of an Existing Concrete Prestressed Bridge

Tommaso Donolato, Neryvaldo Pereira, and José C. Matos

Abstract Reliability is an important factor to determine how safe is a structure. The aim of this study is to use the concept of reliability in order to manage the maintenance and to plan the interventions that could be necessary. The first part includes the calibration of the model, verifying the obtained results. The second part provides a 100-samples nonlinear analysis, considering the statistically important random variables. Each sample is generated considering the mean and standard deviation values of each random variable, using the Hypercube Latin method to couple them. The output is the load factor probability distribution. Using an overload probabilistic curve, the reliability index is computed, according to the Monte Carlo method. The third part illustrates the corrosion effect calculation, using FIB Bulletin 34 guidelines. Once determined the corroded area and the corrosion depth during time, the reliability index is computed, using different time values. The trend of reliability index during time is obtained in relationship with variation of the standard deviation and the load factor values.

Keywords Probability · Risk · Reliability · Corrosion

1 Introduction

During these years, several bridges have collapsed causing huge damages to economy, people life and environment. These collapses are generated by different causes, which are summarized by Fig. 1 [1]. A procedure has been established to prevent these risks and to assure the solidity (safety) of bridges' main structures. A useful tool to reach this aim is the reliability index computation.

The reliability of a structure is its ability to fulfil its design purpose for some specified design lifetime. Reliability is often understood to equal the probability

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J. C. Matos et al. (eds.), *18th International Probabilistic Workshop*, Lecture Notes in Civil Engineering 153, https://doi.org/10.1007/978-3-030-73616-3_38

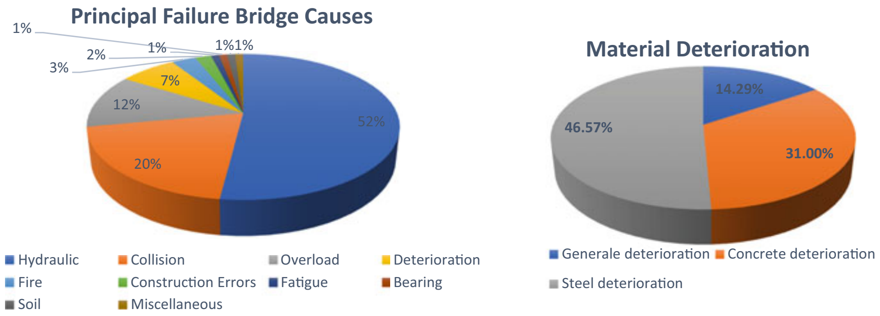


Fig. 1 Bridges failure causes—AASHTO [1]

that a structure will not fail to perform its intended function [2]. Reliability analysis can be applied to evaluate existing structures, assessing their safety and their state, preventing and assuring their correct maintenance. The reliability index can be considered as a rational evaluation criterion. It provides a good basis for decision about the repair, rehabilitation or replacement of the structures. The reliability index provides a methodology to establish the security level. To begin a structural reliability analysis, it's necessary to define a "limit state". The considered limit state in this study is the bending moment limit state, taking into account the middle span cross section, of the biggest span of the studied bridge.

As it can be seen in Fig. 1, deterioration due to corrosion is the one of the most frequent causes of bridge collapsing. The final aim of this study is to compute the corrosion effect, which can affect the reinforcements and can decrease the structural resistance. The corrosion effects are computed using FIB Bulletin 34 guidelines and are reported by the graphs.

During the design and construction phases, all the standards and the requirements are followed, in order to guarantee resistance and durability of the structures. In these phases, the material and geometrical properties can change and be different from the design values [3].

2 Reliability Analysis

The typical performance function, or *limit state function*, can be defined as [2]:

$$g(R, Q) = R - Q \tag{1}$$

The limit state can be designed as the value of the function g, where the limit state is not fulfilled (for example $g(R, Q) < 0$, which corresponds to the structural failure).

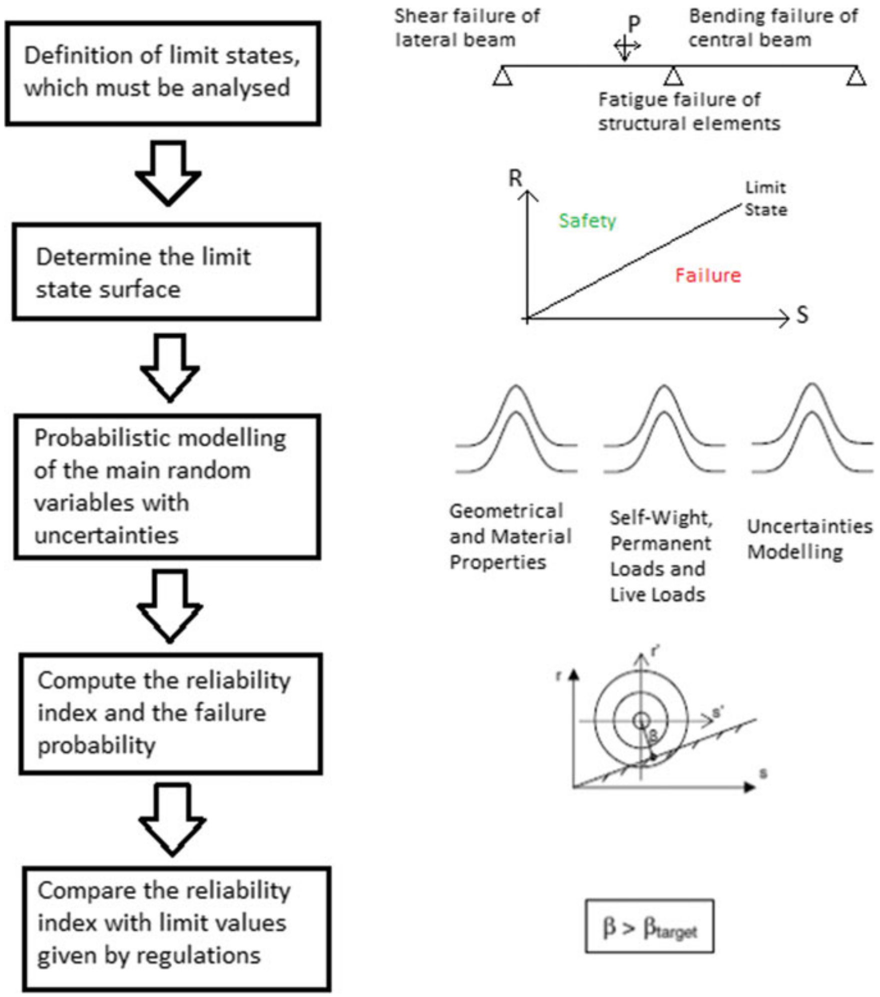


Fig. 2 Reliability analysis flow-charts

This relation can be translated, considering the probability of failure P_f, equal to the probability that the undesired performance will occur:

$$P_f = P(R - Q < 0) = P(g < 0) \tag{2}$$

The reliability index β is the inverse of the coefficient of variation of the function $g(R, Q) = R - Q$. R and Q are two independent random variables. In the case studied, these variables are normally distributed; in this way the reliability index is related to the probability of failure:

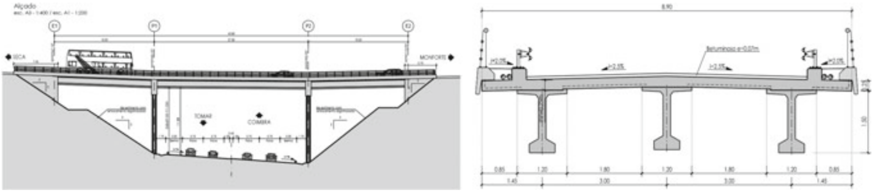


Fig. 3 Studied case geometry—Ascendi [4]

$$\beta = -\phi^{-1}\left(P_f\right) \tag{3}$$

In the following chapter the procedure adopted, is described step by step. A summary can be found in the flow-chart in Fig. 3.

The reliability index is computed considering 100 samples, generated with the help of MatLab®. These 100 samples are based on the Monte-Carlo simulation method. The Monte Carlo method is a special technique used to generate some results numerically, without any physical testing.

2.1 Model Calibration

Considering the case studied, a finite element model is realized using the FEM software DIANA®. The model is verified in order to be sure that the analysis gives correct results. The deck is supported by two piles and the extremizes. It overpasses Tomar-Coimbra road, following the directions of Seca and Monforte (Portugal) [4]. The deck has 3 "I-Type" beams (concrete class C45/55) that are simply supported by the piles (concrete class C30/37). The intersections between slab, beams and piles are all monolithic (special particular to guarantee continuity at the node). The "I-Type" beams are 1.5 m height and 3 m wheelbase and the total transversal length is 8.9 m. They are made in factory with prestressed precast concrete technology. The slab thickness is 0.25 m (concrete class C30/37).

The nonlinear analysis makes possible to compute the capacity curve, in relationship with the load combination used to obtain the most disadvantageous situation for bending moment. A phased nonlinear analysis is adopted to reproduce the stresses of construction stages. The first phase is characterized by the beams simply supported and loaded by self-weight and prestress equivalent forces; the second phase is characterized by a continuous, monolithic structure, loaded with the permanent loads and traffic loads [5–7]. A summary is reported in Fig. 4.

The results are performed using Newton-Raphson Modified method. In Fig. 5, the capacity curve is reported: it can be observed the first elastic behavior, for load factor up to 1.81, and the hardening behavior until its collapse. The numerical results are compared with analytical ones, in order to be sure that the numerical model returns results with an acceptable error percentage.

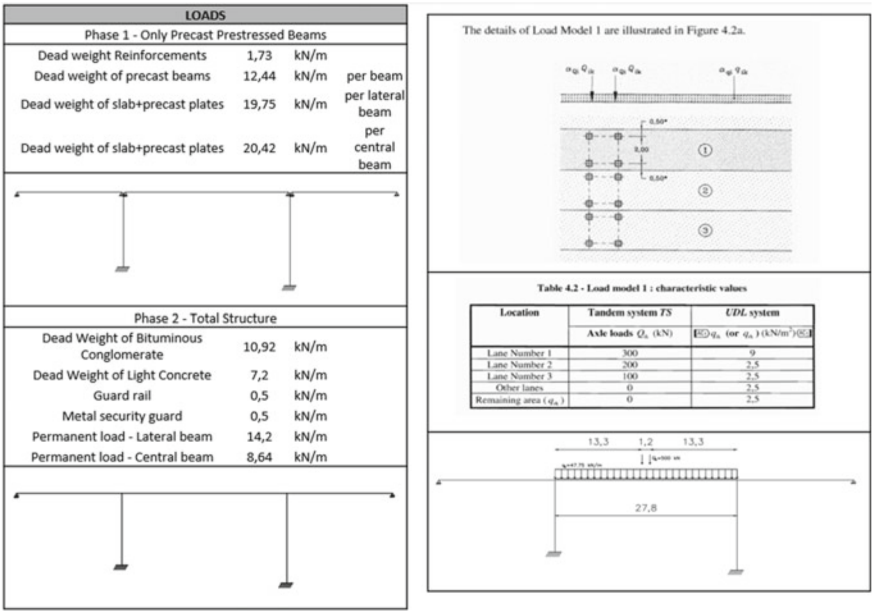
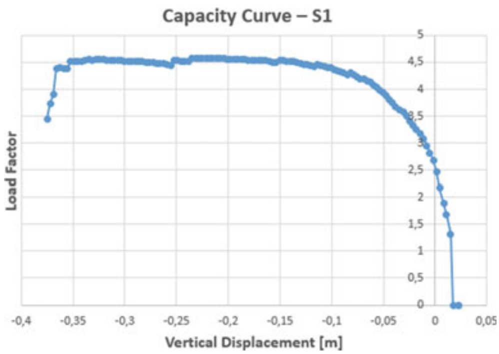


Fig. 4 Load scenarios for phased nonlinear analysis

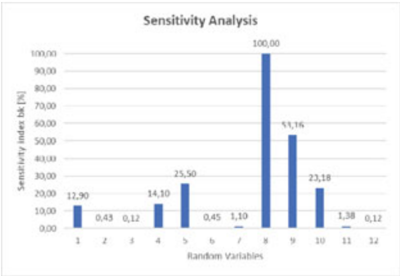


Bending Moment Error	
Numerical [kNm]	17.300
Analytical [kNm]	17.624
Error [%]	1.84

Fig. 5 Model evaluation

2.2 *Probabilistic Analysis*

Once the model is evaluated and verified, the probabilistic analysis can be performed. The first step is to determine the random variables, related to the maximum bending moment calculation (sensitivity analysis). The results, obtained by this analysis, are:



	Material	Random Variable	Name	Average Values	Bias	COV
1	C30/37	Compressive Strength	fcm	38 MPa	1,27	12%
2		Tension Strength	fctm	2 MPa	1,45	20%
3		Young Modulus	Ecm	33000 MPa	1	8%
4		Thickness Slab	e	25cm	1	3,50%
5	C45/55	Compressive Strength	fcm	53 MPa	1,18	9,00%
6		Tension Strength	fctm	2,62 MPa	1,45	20%
7		Young Modulus	Ecm	36000MPa	1	8%
8	A500	Ultimate and yielding stress	fsy fu	560 Mpa	1,12	5,40%
9		Area	A	-	-	2%
10	A1860	Ultimate and yielding stress	fpy fpu	1258MPa	1,04	2,50%
11		Prestress	σp	1087MPa	1	1,50%
12	C30/37 and C45/55	Self-Weight Concrete	γc	25 kN/mc	1,03	8%

Once the random variables are determined, using their mean and standard deviation values [8–10], 100 samples are generated, coupling the variables randomly, considering every value only once. These values are substituted with the respective values of the "mother" finite element model (exported by a data file.dat by the software [11]). This procedure makes possible to have 100 data files, which must be run. A flow chart is reported in Fig. 6 to explain the whole procedure.

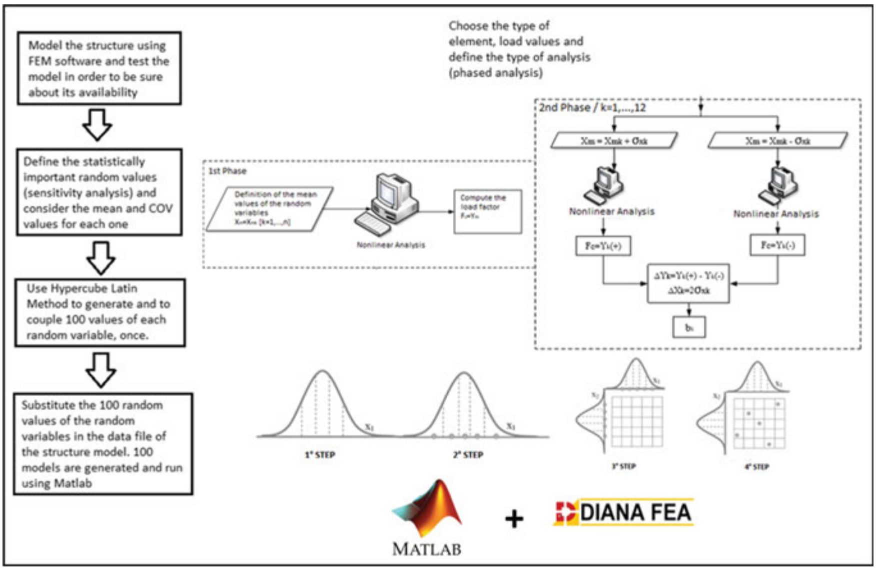


Fig. 6 Probabilistic analysis procedure

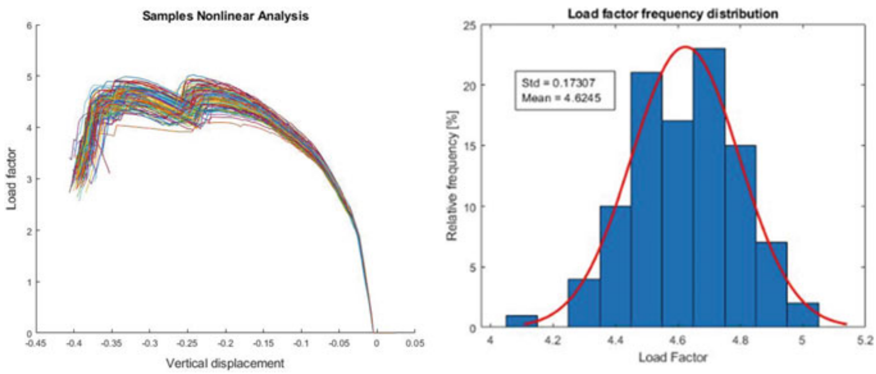


Fig. 7 Probabilistic analysis results

The results of the 100-samples are reported, and they are plotted using a histogram, which considers the relative frequency of occurrence in Fig. 7. The data are fit using a normal distribution in order to use the relation of Nowak-Collins [2].

It can be seen from the load factor-displacements curve the elastic behaviour of the structrure in the first part; then the hardening until the collapse (no convergence) of the model.

The probabilistic load distribution is considered normal too, using a mean value equal to 1 and a standard deviation equal to 0.15 [12, 13].

3 Long Term Effects

The long-term effects are computed considering the corrosion which affects the concrete structures. The corrosion is characterized by two events [14]: the initial phase, ends when the limit state of reinforcement depassivation are reached and the propagation time, phase that is divided in limit states of crack formulation, spalling of concrete cover and collapse through bond failure or reduction of cross section. These periods depend on the exposure classes, which are reported in standards. The typical corrosion process trend is reported in Fig. 8.

3.1 Initiation Time

The initiation phase of the process of carbonation-induced corrosion is marked with carbonation penetration in concrete, and roughly, it finishes with depassivation of reinforcement. Considering an environmental class of exposure equal to XC4, the concrete cover is 65 mm. The needed data to compute the initiation time (4) is determined by literature [14]; in the specific, for the CO_2 concentration different

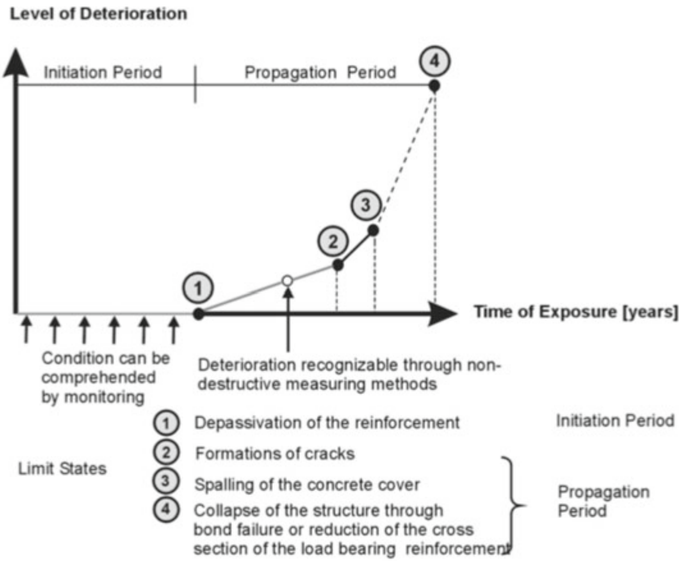


Fig. 8 Corrosion process—FIB Bulletin 34 [14]

experimental studies of Andrade C. are considered. The initial time, taking into account all these hypotheses, is equal to 55.02 years.

$$t_{ini} = \left(\frac{k_{NAC}^2 \cdot k_e \cdot k_c \cdot k_a}{c^2} \cdot t_0^{(p_{dr} \cdot ToW)^{bw}} \right)^{\frac{1}{(p_{dr} \cdot ToW)^{bw} - 1}} \tag{4}$$

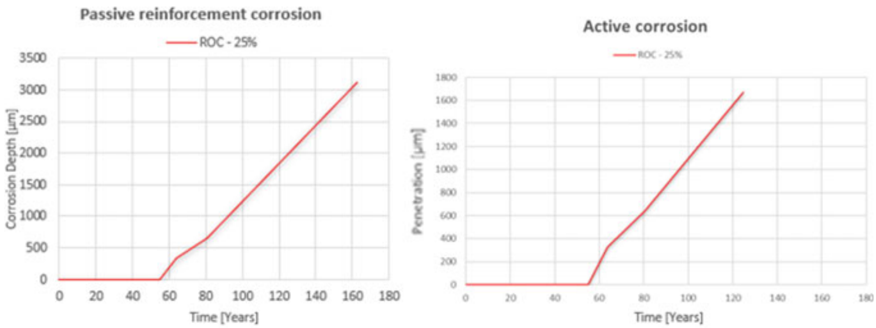
3.2 Propagation Time

There are several mathematical models and empirical formulas which describe the propagation of corrosion. Several studies make possible to predict corrosion depth and residual diameter [15–17].

In the FIB Bulletin 34 there are three different limit states for corrosion:

1) Cracking limit state (SLS)
2) Spalling limit state (SLS)
3) Collapse limit state (ULS).

For each one of these limit states, the corresponding time is computed. The procedure follows the FIB guidelines [7]. The corrosion effects are computed for passive and active reinforcements both. The results are reported in Fig. 9.



ROC 3 [25%]			
t	P(t)	Φ(t)	ξ
0	0	0	0
55,02	0	0	0
63,36	317	24,37	0,949
80,70	659	23,68	0,899
162,93	3125	18,75	0,563

ROC 3 [25%]			
t	P(t)	Φ(t)	ξ
0	0	0	0
55,02	0	0	0
63,64	328	12,69	0,94
80,46	639	12,07	0,89
124,7	1670	10,01	0,56

Fig. 9 Corrosion effects

The corroded area limit is given by the regulations reported in literature [7]; it depends on the robustness class, which is related to an allowed percentage of corroded area. In this study, the allowed percentage is 5% (ROC3).

In order to perform reliability analysis, considering the probabilistic distribution reported on FIB Bulletin (the propagation time, referred to a specific environment, has a lognormal distribution), Monte-Carlo simulation is run, in order to obtain a lognormal distribution for the propagation time (cracking, spalling and collapse limit states). From the propagation time, taking into account the corrosion relationships between time and corrosion depth, the corroded area lognormal distribution is determined. The results are reported in Table 1.

The corroded area refers to a single reinforcement bar (25 mm for passive reinforcement and 15.2 mm for active reinforcement) and the MatLab® script assign the total corroded area.

Table 1 Propagation time

Propagation time – Passive reinforcement			
Cracking		Spalling	
A(t)	466,111 mmq	A(t)	442,912 mmq
STD	7897 mmq	STD	12,972
Propagation time – Active reinforcement			
Cracking		Spalling	
A(t)	113,541 mmq	A(t)	114,5843 mmq
STD	6652 mmq	STD	6,717,003 mmq

4 Long Term Probabilistic Analysis

The final step of this study is to perform a long-term probabilistic analysis. The procedure is almost the same adopted for the probabilistic analysis in Sect. 2.2. The non-corroded area values are substituted by the corroded ones (with the help of a MatLab® script) and the previous standard deviation values too. Using this new data, a 100-samples analysis is conduced for each value and the output is, as before, the load factor probabilistic distributions. The final output is the trend of the reliability index, depending on the time.

4.1 Long Term Reliability Index

The reliability index is computed considering the relationship [2]:

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \tag{5}$$

μ_R is the resistance mean, μ_Q is the stressing loads mean, σ_R is the resistance standard deviation, σ_Q is the stressing loads standard deviation.

The purpose is to define how the reliability index decreases during time. In relation to the reliability index, the standard deviation is plotted too, in order to understand how large the error involved in the reliability analysis is. The reliability index is computed considering load and resistance uncertainties [8].

The obtained results for the reliability index are reported, depending on time, on Fig. 10. In the end, the standard deviation is plotted in relationship with time. The purpose is to observe how reliable the obtained results are. If the standard deviation increases too much, the probabilistic analysis can't be considered, and some experimental campaign must be performed in order to remove as many uncertainties

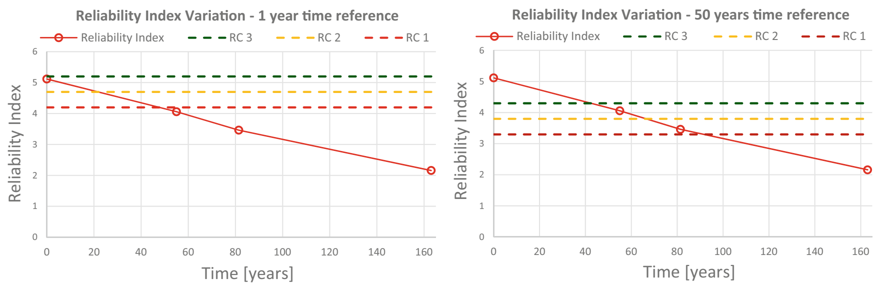


Fig. 10 Reliability index—EC0

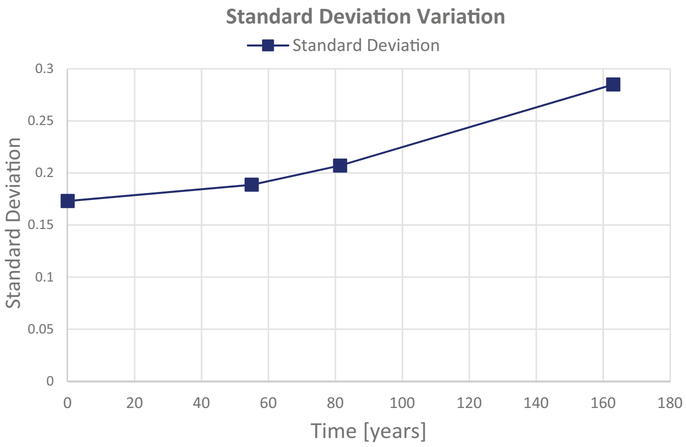


Fig. 11 Standard deviation during time

as possible. The standard deviation values are plotted against time in Fig. 10 in order to determine the availability of the analysis.

As expected, the reliability index trend decreases during time, due to corrosion progress. Using this method, it can be possible to plan every maintenance intervention in order to prevent any collapse events, which would cause relevant damages to people life, economy and environment. Using the values reported in Eurocode [5] the limits of acceptable reliability index are fixed, in dependance of reference time. Shorter is reference time, more rapidly the maintenance interventions are required.

In the end the standard deviation is reported:

The standard deviation increases during time because the uncertainties linked to the process are more significant. Reducing the different uncertainties of the model the results can be improved.

5 Conclusions

As expected, the reliability index decreases in time, but the values reach a critical value only at the end of the structural life. The critic event corresponds to the complete loss of the reinforcements and, consequently, to an important decrease of resistance of the structure. In the graph, the limit values of acceptable reliability index are reported in order to underline the critic states, where the probability of failure assumes a value which is not acceptable. This procedure makes possible to plan the needed maintenance interventions.

The standard deviation increases during time. This is real because in the model and in the introduced characteristics, the uncertainties about the model, the materials, the reliability index calculation are different. In the analysis, these uncertainties are not taken into account. If they are considered, the reliability index might be lower, and the structure might reach critic situation earlier.

Acknowledgements This work was co-financed by the Interreg Atlantic Area Programme, through the European Regional Development Fund, under the project SIRMA (Grant EAPA_826/2018).

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