Robustness and Life-Cycle Analysis (LCA) of Structures and Infrastructures

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Abstract

Life-cycle analysis (LCA) seeks for optimization of design/ construction /maintenance/deconstruction costs, either economic, social or environmental costs. Nowadays, advanced optimization techniques are available. However, some aspects related to the decision making during the maintenance policy are difficult to be included in the objective function to be optimized. This refers to shortage of required funding or grouping of maintenance works in different assets to be deployed at the same time, etc. In this paper, the measure of the robustness of the structure/infrastructure as a way to take into account those aspects related to the maintenance/repair work is introduced. The main idea is to relate the robustness of the structure/infrastructure to their ability to adapt to scenarios beyond those proposed by the LCA as the solution of the optimization problem. As a consequence, the proposed measures of robustness become of relevant importance in a "real world" and not only "theoretical" application of a life-cycle analysis strategy to existing infrastructure assets.

Keywords: robustness, maintenance, damage, structural condition, reliability

1. Introduction

Because of limited available funding, an important issue when dealing with the life-cycle analysis (LCA) of structures and infrastructures is to decide on the urgency of the repair/strengthening when there are not sufficient funds to perform all the interventions necessary in the network as derived from the optimization problem. In other cases, the practical application in the "real world" of the maintenance activities at different times, as proposed by the management system, suggests to wait some additional period of time in order to get a minimum amount of work to be deployed in the managed infrastructure in just one main activity. This produces important savings. In both cases, the decisions result on the no application of the maintenance/repair/strengthening interventions at the exact time proposed as the output of the life-cycle analysis. In both cases, a crucial question arises: How postponing a specific maintenance/repair work can increase the repair costs disproportionately and/or affect safety in a relevant way? The sensitivity of the assets to this issue seems to be dependent on the material, structural configuration, type of damage, redundancy,…

This paper proposes the measure of the robustness of the structure/infrastructure as the possible answer to this question. To this objective, two possible approaches are defined and, therefore, two indicators of structural robustness are proposed. The first one can be applied during the condition assessment of the structure and is able to quantify the effect of the damage/deterioration when discovered during inspection on the overall system behaviour and not only at the member level. This may suppose a better condition rating than the one initially proposed by the inspector. In the second case, robustness is defined as a structural property related to the tolerance to damage and is measured by the average performance considering all possible damage scenarios that the structure/infrastructure may see if not immediately repaired. In both cases, higher robustness will derive on higher adaptability of the system to postpone the necessary interventions.

2. Robustness

As presented in Cavaco et al. (2013) and Anitori et al. (2013), several definitions of robustness exist. The approach presented in this work is based on the most general definitions of robustness as proposed by Knoll and Vogel (2009): "Robustness is the property of systems that enables them to survive unforeseen or unusual circumstances" and Baker at al. (2008): "Robustness is taken to imply tolerance to damage from extreme loads or accidental loads, human error and deterioration". Again, from the many indices proposed to quantify robustness (Cavaco et al. 2013), those proposed by Ghosn and Moses (1998) and Cavaco et al. (2013) will be considered here. The first one points to the direction of defining robustness as the tolerance or the ability to adapt to unforeseen actions, and the second in the sense to the tolerance or the ability to adapt to deterioration mechanisms.

2.1 Robustness and condition rating

The first measure of robustness is used to develop a methodology for the correction of the condition rating assessment, making it dependent on the system behavior. Robustness and redundancy concepts that have been used in the past for structural design of new bridges and for assessment of existing bridges are in this work extended to condition rating, which represents the first step of bridge health condition evaluation of bridges life-cycle.

In the field of bridge condition assessment, condition rating based on visual inspection is the first step before more accurate evaluation processes and maintenance decisions are proposed. Bridge owners, like privates or public administrations adopt different policies for the condition appraisal of their infrastructures depending on many factors consisting basically in the minimization of monetary cost and maintaining the safety level. The main objective of this first stage of inspection is to rate the bridge condition and to finally give priority of maintenance to those bridges that result in weaker conditions. Although different owners have different rating rules for their bridges, the engineering judgment and in field experience of the inspectors are mandatory requirements for a reliable condition rating, and a complete automatic bridge rating does not seem feasible yet. On the other hand, research effort should go towards a reduction of the subjectivity of these practices.

Tenzera (2012) pointed out that different expert inspectors can label the same damage with a different rating and that the inspector's judgment can lead to an important dispersion of bridge rating results. Moore (2001) carried out a study on visual inspection reliability in USA concluding that dispersion on the results of bridge rating evaluated by different inspectors over the same bridge present dispersion due to human subjectivity and function of some external environmental reasons like presence of traffic, structural accessibility, wind speed etc. These research efforts highlight the need for an improvement of the inspectors training and of the condition rating procedures. At the same time, it would be unwise introducing new and untested measures, neglecting the experience gained by engineers' community during the years. For this reason the present research points into the direction of the correction of typical bridge condition rating as a result of visual inspection. The procedure is theoretically applicable to every condition ranking based on a numerical output.

Typically, conventional condition rating procedures give little importance to the system behaviour and only look to the local consequence of an existing damage. However, it seems logical to think on the possibility of recalculate the condition rating based on the effect that the local damage may produce in the overall performance. The recalculation should take into account the robustness of the structure in the sense presented earlier, i.e., "the ability to survive unforeseen or unusual circumstances" not devised during the design. Therefore, the objective is to propose a so-called "robustness factor". With this factor, the inspector in charge of the condition rating, will be able to decrease marks of those bridges that are less robust and increase the mark of more robust bridges. More robust bridges will see decrease their position in the ranking of the most urgent structures to be repaired. In other words, more robust bridges can wait longer an upgrading intervention because the probable failure of the member where the damage is located is going to affect less the bridge safety than if the failure occurs in a less robust bridge. The robustness factor is associated with a structural type, configuration and damage and can be collected in a damage inventory.

Among the different parameters that an inspector can consider as determinant in guaranteeing the safety of structures, little attention is often given to the structural type. A structural type is defined as a common design with a well-known structural behaviour. Typical structural types for bridges are I-beams prestressed concrete bridges, steel I beams bridges, box prestressed concrete beam bridges, box girder bridges etc. Every structural type is characterized by a different set of configurations like number of beams, their spacing, span length etc.

The methodology proposed in this paper is meant to calculate a coefficient that modifies a typical condition rating, obtained with the standard inspection procedures, taking into account the bridge structural type and configuration. In fact, the determination of the structural type is an easy task and an objective attribute. The reason for such correction is due to the fact that encountered damage may affect the capacity depending on the structural configuration. The way to take into account the bridge robustness in the condition rating assessment is based on the redundancy measures proposed in Ghosn and Moses (1998). The deterministic measure and criteria used by Ghosn and Moses (1998) and Liu et al. (2000) were based on probabilistic models (Anitori et al 2013). In particular, are proposed three reliability measures of redundancy which are defined in terms of relative reliability indices as:

$$\begin{aligned} \Delta\beta_u &= \beta_{ult} - \beta_{member} \\ \Delta\beta_f &= \beta_{funct.} - \beta_{member} \\ \Delta\beta_d &= \beta_{damaged} - \beta_{member} \end{aligned} \tag{1}$$

where β_{member} represents the reliability index related to the failure of one member; $\beta_{ult.}$ the reliability index related to the collapse of the originally intact structure; $\beta_{funct.}$ the reliability index related to the loss of functionality of the structure; $\beta_{damaged}$ the reliability index related to the collapse of the damaged structure. These relative reliability indices, or alternatively the corresponding deterministic load factors, give a measure of the additional safety provided by the system compared to the safety against first member failure (figure 1).

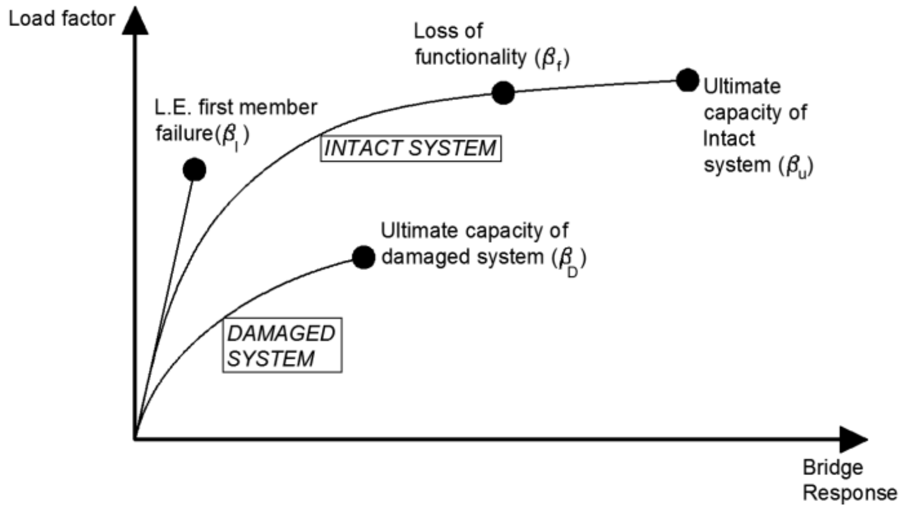

Figure 1. Capacity curve for a generic bridge system in its intact and damaged conditions. The four dots indicate the points in which reliability indices or load factors are calculated.

In this study, the focus is on precast concrete bridge beams deteriorated by corrosion of prestressing steel. In the case of the corrosion of steel strands in prestressed I-beams, as part of multi-beam bridges, the pair “beams + upper slab” are expected to behave as a structural system and to be able to redistribute the load after the failure of one of them.

The corrective factor or “robustness factor” proposed in this research is based on the reliability index as the measure of performance and takes into account the relative importance of them. As example, the bridges considered in this study have been rated following the rules of Casas (2007), but any condition rating procedure with a final numerical value may be of application. The condition rating methodology is based on the following steps (Anitori et al. 2013):

1. Identification of the main parts of the bridge (abutments, beams, bearing pads etc.)
2. Estimation of a condition index ($I_{M,k}$) for each member *k*.
3. Estimation of the bridge condition index (I_C) based on the condition of each member.

The condition index at member level is calculated by the selection of the following three partial indicators:

a) **Severity of the damage.** It is based on the damage type and is provided by the inspector based on a set of damage intensity.
b) **Intensity of the damage.** It represents the state of degradation of the damage considered.
c) **Extension of the damage.** It represents the portion of the member affected by the damage.

The robustness of the bridge is introduced in the following way:

$$I_C{}^* = \phi_R \, I_C \qquad (2)$$

where I_C is the condition index obtained by standard conventional method, ϕ_R is the robustness factor and I_C^* is the corrected condition index accounting for the bridge robustness. A higher condition index means the bridge is in better condition and repair is less urgent. The robustness factor, ϕ_R, reflects the redistribution capability of the bridge associated with a damage condition (e.g. corrosion of a main member). Their values are obtained via calibration process. The objective of the calibration is to find a coefficient that modifies the condition rating of a generic bridge to reach the performance of a target bridge. The relative reliability index of the target bridge is labelled as $\Delta\beta_{d,target}$.

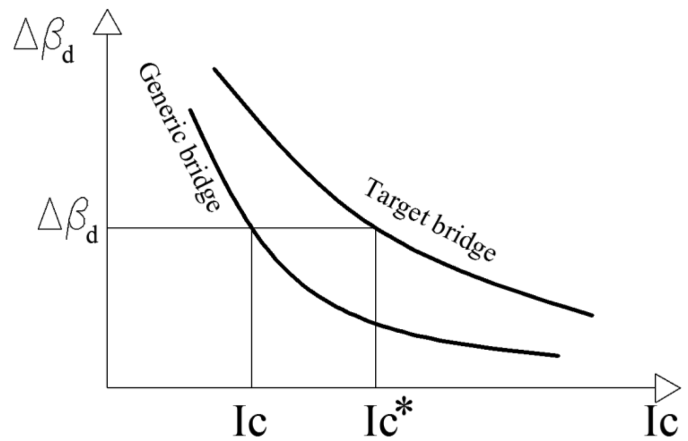

Figure 2. Calibration process in which the condition index of a generic bridge is corrected to meet the performances of the target bridge described by the damage relative reliability index.

The procedure of calibration consists in the following steps (figure 2):

1. Calculate $\Delta\beta_d$ (relative reliability index in the damaged state in equation 1) for a generic bridge affected by a corrosion level δ_d
2. Calculate I_C according to standard rating method
3. Select a guess value for ϕ_R and calculate $I_C* = \phi_R\ I_C$
4. Calculate the corrosion level δ_d* associated with the modified condition index I_S*
5. Calculate the new $\Delta\beta_d*$
6. Repeat step 1 to 6 until $\Delta\beta_d* = \Delta\beta_{d,target}$.
7. The robustness factor ϕ_R found is the correction that must finally be used for the final condition rating of the bridge.

2.1 Robustness and deterioration

The increase in investment needs for the repair, retrofitting and replacement of existing infrastructures in most developed countries leads to the need to achieve tools for assessing the impact of present and future deterioration on safety. Deterioration is often a localized damage, as some elements can present significant loss of performance, as others are almost intact. Due to this, it is expectable that, in many structures, the loss of resistance of one element due to deterioration can be balanced by the existing resistance reserve of other elements. To evaluate the safety of large sets of similar structures, a robustness indicator can be used to quantify, in a simplified manner, the loss in resistance under different deterioration scenarios. This analysis must consider the redundancy of the structure.

Robustness assessment is usually associated with damage caused by accidental actions or unforeseen load scenarios, like explosion, fire or impacts. However, damage could also arise slowly in time, from aging of structures, as induced, for example, by environmental aggressive agents. As presented above, this is one of the definitions done by Baker and his co-authors (Baker et al. 2008). In this context, it is of great interest to develop suitable life-cycle measures of structural robustness with respect to a progressive deterioration of the structural performance due to a continuous action more than a sudden scenario.

A new robustness index has been proposed by Cavaco et al. (Cavaco 2013, Cavaco et al. 2011, Cavaco et al. 2013) for the evaluation of structures under likely exposures, including deterioration. The robustness index is defined by the average performance of the structure subjected to damage varying over its total spectrum. In order to avoid lack of sensitivity of the robustness measure, both structural performance and damage can be normalized. Therefore the robustness index, R_d, for a specific performance and damage can be given by the area below the curve which corresponds to the normalized structural performance, $f(D)$, function of the normalized damage D, as (figure 3):

$$R_d = \int_{d=0}^{d=1} f(x)dx \tag{3}$$

The complexity of the proposed measure depends on the indicator used to measure the structural performance *f*(*D*). If it is a simple indicator, then the proposed index is in fact quite simple. For example, if the performance indicator is the structural load carrying capacity, *f*(*D*) would be obtained through the ratio $R(D = d)/R(D = 0)$ where *D* is any damage scenario resulting from an exposure. In this case R_d would represent the average normalized load carrying capacity of the damaged structure. On the other hand, if *f*(*D*) is based on a more complex performance indicator, such as the reliability index, the model grows in complexity. Equation (3) corresponds to considering that robustness is defined by the area under the normalized performance for the damage levels between 0 (intact) to 1 (total damage). In this sense, a structure for which any damage causes a complete loss of performance is considered not robust (curve A in Figure 3), as a structure for which no reduction in performance occurs for any damage level corresponds to full robustness (curve E in Figure 3). Real structures will correspond to situations between these two extremes (curves B, C and D) and the shape of the curve will show the susceptibility of the structure to deterioration. For example, figure 4 shows a performance function calculated as the ratio between the bond strength in the corroded condition of a reinforced concrete beam (X_p being the corrosion level) and the bond in the intact condition. This function is calculated for different intensities of the corrosion in the RC system.

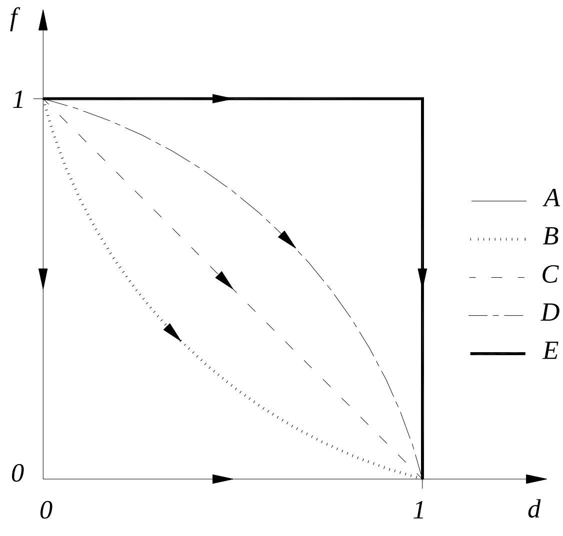

Figure 3. Robustness assessment. Normalized structural performance *f* as a function of the normalized damage *d*

As the proposed measure is based on the normalized performance, the proposed index can not reflect if the structure in intact or damaged state is able to perform according to the design objectives. The proposed measure reflects only if the structure has the ability to sustain damage by maintaining a consistent performance. Therefore, even a structure that does not met design criteria can be considered as robust, if average performance on the damage state do not differ much from the performance in the pristine state. However, if the performance level is of concern, the proposed measure can be used without normalization.

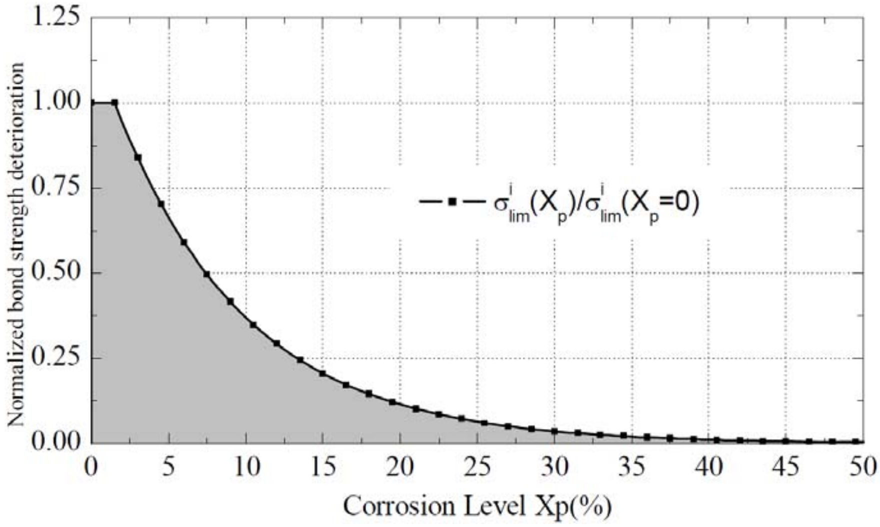

Figure 4. Robustness index represented by the shaded area under the function f. Deterministic measure

The proposed index (equation (3)) has a direct impact on the life-cycle assessment (LCA) of deteriorating bridges. In fact, curve B in Figure 3 (lower robustness index) means that deterioration affects more rapidly the bridge performance and safety than curve D (higher robustness index). Therefore, a higher robustness index for a bridge or group of bridges indicates that the maintenance/repair works to be undertaken can wait longer (in the case of lack of funding) than in the case of another type of bridges with lower robustness index. This measure of robustness can be directly linked to the urgency of the repair. On the other hand the robustness index proposed here may also serve to optimize the maintenance policies. In fact, just to apply a repair work on an existing bridge in time *t* and a similar repair work in time *t+t1* in another bridge, is normally more expensive to wait until time *t+t1* and carry out both repairs at the same time. Of course, for that to be feasible, the bridge requiring repair at time *t* should be able to wait without jeopardizing safety. A higher robustness index for this bridge will mean a higher tolerance to waiting.

3. Application

Prestressed concrete I-beams bridges are considered in this example of application of robustness factor because they represent one of the most common structural type in bridge engineering and, according to Ghosn and Moses (1998), many configurations of this deck type are considered redundant. The robustness factor is strongly correlated to redundancy measures (Eq. 1). However, any kind of deck in which redistribution of load carrying capacity after a local failure are produced, can be analyzed by this procedure.

The structures taken into exam are simply supported multi-beam bridges. The beams are AASHTO types prestressed by 1800 MPa low-relaxation steel. The beams' concrete strength is 50MPa while the slab concrete strength is 35MPa. A total of 120 different configurations of bridges have been considered to cover all the typical designs of multi-beam bridges. The span length ranges from 13 to 45m, the beam spacing ranges from 1.2 to 3.6 m and the number of beams ranges from 4 to 10. Figure 5 shows an example of bridge configuration of a 30 m span, 6-beams deck.

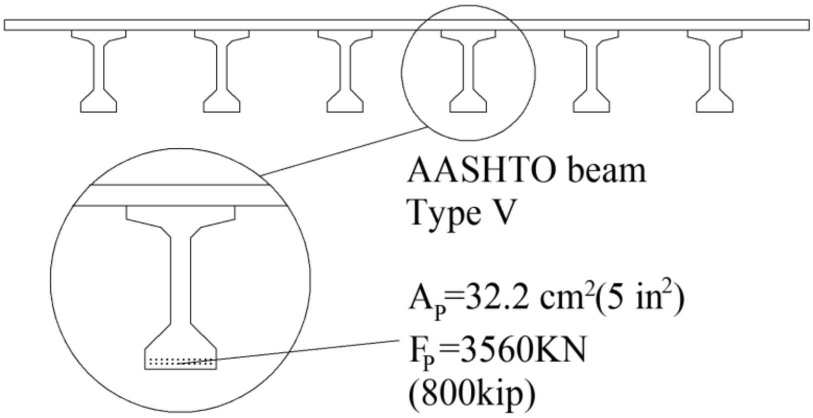

Figure 5. Deck configuration of the considered bridge type.

The deterioration considered in this example is corrosion of the prestressing steel. The effect of this process on structural steel, in general, is the transformation of a portion of superficial material into expansive oxidized products with poor mechanical properties. This reflects in a loss of steel area, a loss of ductility, and a decrease of strength. Furthermore the degradation of the steel surface by the oxidation products leads to a decrease of the bond strength.

The outputs that are necessary to calculate the robustness measures previously pointed out, are the vertical load factor that leads to the failure of the first member and the load factor that leads to the system collapse. The latter is calculated in the bridge intact configuration and in its damaged states. In order to calculate these load factors, a nonlinear static analysis is performed based on a step-by-step Newton-Rapson algorithm. A plane frame grillage is used to calculate the capacity of the bridge as explained in Anitori et al (2013).

The damage is modeled considering the following assumptions:

1) The reduction in steel ultimate strain because of corrosion is calculated according to Vergani (2010) and Biondini (2011):

$$\varepsilon_{su} = \begin{cases} \varepsilon_{su0} & ,0 \le \delta_s < 0.016 \\ 0.1521\delta_s^{-0.4583}\varepsilon_{su0} & ,0.016 < \delta_s \le 1 \end{cases} \tag{4}$$

where ε_{su0} is the steel ultimate strain of the undamaged strand and δ_s is the corrosion penetration.

2) The area reduction depends on the corrosion penetration δ_s parameter which is related to the steel mass loss and to the coexistence of general and pitting corrosion (Biondini 2012).

3) The bond ultimate strength is decreased using the following experimental equation:

$$\frac{\sigma_{\lim}(X_p)}{\sigma_{\lim}(X_p = 0)} = \begin{cases} 1.0 & if \quad X_p \leq 1.5\% \\ 0.192e^{-0.177X_p} & if \quad X_p > 1.5\% \end{cases} \tag{5}$$

In order to be able to extend the results to the whole population of prestressed concrete beams some assumptions must be set in the analysis:

1) Only the strands in the proximity of the cover are considered under corrosion process.
2) A portion of strands are modeled to contribute only in the central part of the span to simulate the effect of strand intubation which is usually adopted to reduce the negative moments in the support region. An intubation length of 50% of span length is considered for 30% of the total amount of strands.

In order to associate the strand degradation to a visual indicator like crack width an empirical equation is utilized (Vidal 2004, Anitori et al. 2013).

The robustness indices found as result of the calibration process are higher than one whenever the damaged condition of a given bridge leads to a structural performance lower than the target one. In other words, the structural consequences of member failure are more severe than the consequences of member failure in the bridge used as a base for the calibration in terms of relative reliability index. On the other hand, when a bridge presents robustness factors smaller than one, it means that the consequence of a member failure in a certain damage state is less relevant than in the target bridge. Figure 6 shows the sensitivity of the robustness index to the number of beams and figure 7 to the beam spacing. Figure 6 shows that large beam spacing is more likely to provide a benefit to the evolution of the corrosion over the system behavior, while small beam spacing can lead to the opposite. Furthermore, there may be configurations of bridges insensitive to the number of beams because of the large beam spacing that make inefficient the load migration to the intact members after a member failure (see tables 1 and 2).

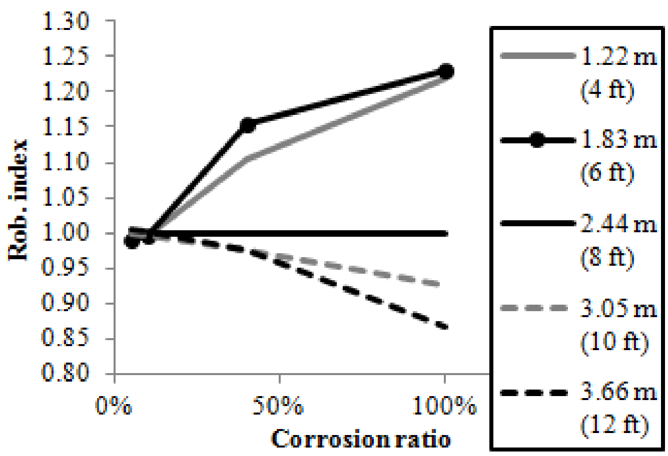

Figure 6. Robustness index vs. corrosion ratio: sensitivity of the bridges to beam spacing

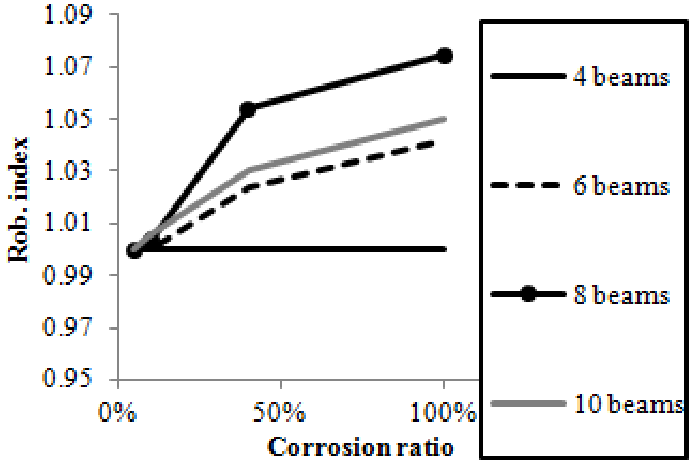

Figure 7. Robustness index vs. corrosion ratio: sensitivity of the bridges to number of beams.

Table I: Corrosion, crack width and robustness factor (14 m span length, 4 beams, 1.22 m beam spacing).

corr. %	crack width (mm)		ϕ_R
5	0.2		0.99
10	0.8		1.00
40	4.3	spalled	1.10
100	11	spalled	1.22

Tables 1 and 2 show that different bridge configurations lead to different robustness factor evolution versus the corrosion ratio. This is directly related to the capability of the bridge to redistribute load after the failure of a member. These two tables show a proposal of implementation of the robustness indices in a condition rating manual or damage database for visual inspection. In the tables, the crack width and corrosion mass loss should be used by the inspector to select a commensurate robustness factor depending on the refinement of the inspection data.

Table II: Corrosion, crack width and robustness factor (14 m span length, 12 beams, 3.66 m beam spacing).

corr. %	crack width (mm)		ϕ_R
5	0.2		1.15
10	0.8		0.9
40	4.3	spalled	0.88
100	11	spalled	0.87

Tercenas bridge (see Figure 8) is a three-span continuous reinforced concrete bridge located in the Center of Portugal, near the seaside. The bridge, built in 1968, shows clear signs of advanced deterioration, raising concerns on its safety (figures 9 and 10). The bridge is composed of three spans, two lateral 18.60 m long spans and a 22.8m long main span. The bridge is supported by abutments in the margins and two piers located in the riverbed. Due to the poor soil quality, all supports are founded on piles.

Figure 8. Tercenas bridge lateral view

The cross section of the bridge is composed by four reinforced concrete girders with 1.25m cross-section depth. The girders width is variable between 1.10 m, over the supports, and 0.50 m at 1/3 of each span. Over the supports and at 1/3 of each span, a crossbeam 0.30 m wide and 1.25m high was built (figure 9). Accordingly to the bridge original design plans, the top reinforcement of the longitudinal beams is composed by one layer of 16Φ25. The bottom reinforcement is composed by three layers of 5Φ25 in the lateral spans. In the center span, a more superficial layer of 6Φ25 is followed by a 5Φ25 reinforcement layer.

A reliability-based assessment of the bridge was carried out for different levels of corrosion in the different beams (Cavaco 2013). Figure 11 was obtained giving the normalized performance, measured in terms of the bridge deck reliability, as a function of the normalized damage, considered as the corrosion level on the longitudinal bottom reinforcement. In the first curve, both reinforcement and de-bonding effects were considered while in the second curve the later has been neglected.

Figure 9. Bridge deck underview

Figure 10. Corrosion in beam 1

The objective of the robustness index proposed by Cavaco et al. is to be more adequate to structures under deterioration and, as a consequence, to be used on a life-cycle analysis framework for the management of existing structures. It should be pointed out that this application does not need an absolute value of the robustness index and just a relative measure can be used to decide on possible alternatives. This becomes also an advantage of the proposed index because to derive target values of robustness is a very difficult task (Cavaco et al 2013). This is the matter in many structural codes, where the requirement for design of robust structures is claimed, but where no indication on the acceptable level of robustness is provided. Although corrosion was considered to affect only the bottom and most external of two longitudinal reinforcement layers, the bridge deck showed limited safety decreasing (β from 5.2 to 3.8 for corrosion levels ranging from 0 to 100 %). Robustness resulted in R = 76%, which correspond to a mean bridge deck reliability of β = 3.95, and shows the structural system to tolerate corrosion without large impact on safety. Bond deterioration was achieved as the main deterioration factor causing safety reduction in the initial corrosion stages, in particular for corrosion levels below 15%. The effect of reinforcement area reduction is only relevant for more advanced corrosion levels. From Figure 11 it becomes evident that neglecting de-bonding effect results on a higher robustness index. Assuming that a reduction of sectional area of steel in a particular point of the bar, but maintaining a good level of bond in the rest of the bar is a feasible damage scenario for a RC structure affected by pitting corrosion, we may conclude from Figure 11 that RC structures affected by pitting corrosion present a higher robustness than those affected by general corrosion. As a consequence, initially, from the point of view of robustness, priority should be given to the repair/strengthening of the later. However, we should also take into account that for a same period of time, the loss of sectional area in reinforcing bar because of pitting corrosion is much higher than for generalized corrosion. This works in the opposite way, i.e. priority should be given to bridges affected by pitting corrosion because speed of corrosion. The final decision has to consider both effects in the affected bridges.

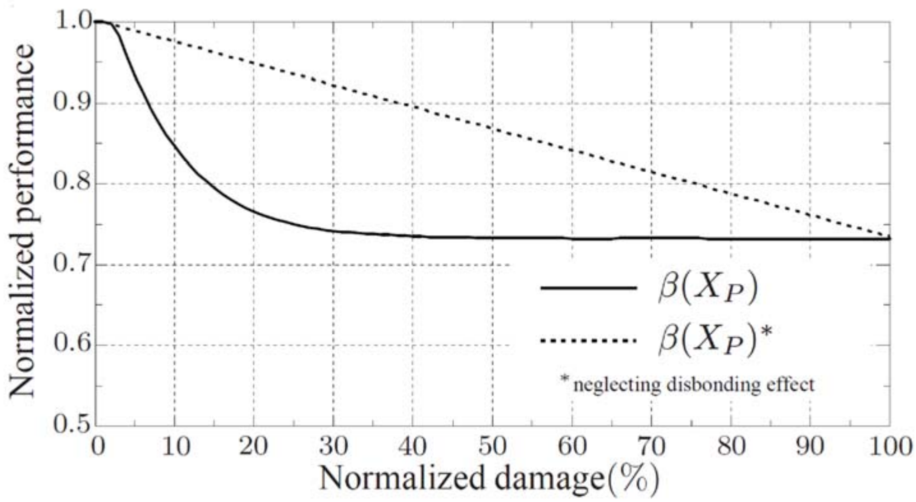

Figure 11. Normalized bridge deck performance as a function of the normalized damage.

4. Conclusions

This paper proposes a method to consider robustness in the condition rating process executed by inspector engineer and in the performance loss because of deterioration along service-life. In both cases, the objective is to build a decision tool to decide about the postponing of a specific repair work.

In the first case, the method assigns a robustness factor, decreasing the estimate of the condition rating if the bridge under inspection is not robust, while an increase of the condition index is allowed for robust structures. The proposed robustness factor is calibrated to provide consistent system reliability levels for all bridge configurations.

A preliminary chart of robustness factors is proposed based on beam spacing and number of beams for different condition rating as would be obtained during a standard bridge inspection process.

The proposed robustness factor in condition rating is of particular interest for an inspector when a group of bridges present similar damage but they have different structural configuration. In those cases, typical condition rating would rate all the bridges with the same mark, therefore being all bridges equally candidates to be repaired. However, accounting for robustness, through the proposed robustness factor, can help to set priorities of intervention managing the repair schedule. The methodology is applied to the loss of performance due to corrosion of the prestressing steel strands, although it could be applied to other types of degradation processes like cyclic fatigue, to other bridge elements as piers in the case of scouring of the foundation, and to other structure/infrastructure assets too.

In the second case, a robustness index is proposed to measure the influence of deterioration propagation in the loss of performance. Again, when a group of bridges present similar damage but they have different robustness index, those with a higher robustness index may wait longer until intervention is carried out, therefore also helping to rearrange the maintenance/repair agenda. This has been presented in the example in the case of a reinforced concrete bridge affected by corrosion of the reinforcement. However, the method is general enough to be applied to other deterioration processes as well as to other structures/infrastructures.

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References

Anitori, G., Casas, J.R., Ghosn, M. (2013) Correction of the condition rating of bridges based on system behavior and reliability. *Proceedings of ICOSSAR-2013, New York, USA*

Baker,J.W., Schubert, M., Faber, M.H.(2008) On the assessment of robustness. *Structural Safety*, vol. 30, No. 3, pp. 253-267

Bhargava, K., Ghosh, A., Mori, Y., Ramanujam, S. 2007. “Corrosion-induced bond strength degradation in reinforced concrete analytical and empirical models.” Nuclear Engineering and Design.

Biondini, F. (2011) Cellular Automata Simulation of Damage Processes in Concrete Structures, *In: Soft Computing Methods for Civil and Structural Engineering, Y. Tsompanakis and B.H.V. Topping, (Eds.), Saxe-Coburg Publications, Stirlingshire, Scotland, Chapter 10, 229-264.*

Biondini, F., Vergani, M. (2012) Damage modeling and non-linear analysis of concrete bridges under corrosion. *Proceedings of the sixth international IABMAS conference, IABMAS-2012, Stressa, Italy*

Casas, J.R. (2007) *Proposal for the condition rating of bridges in the highway trunk network of Catalonia*. Interim Report. Technical University of Catalonia. Civil Engineering Department. Barcelona

Cavaco E., Neves, L., Casas, J.R. (2011) A reliability-based measure of robustness for concrete structures subjected to corrosion. *Proceedings of 11th International Conference on Application of Statistic and Probability in Civil Engineering, ICASP-11. Zurich*

Cavaco, E. (2013) *Robustness of corroded reinforced concrete structures*. Ph.D. Thesis, Universidade Nova de Lisboa, Lisbon, Portugal.

Cavaco, E., Casas, J. R., Neves, L., Huespe, A. E. (2013) Robustness of Corroded Reinforced Concrete Structures. A Structural performance approach. *Structure and Infrastructure Engineering*, vol. 9, No. 1, pp. 42-58

Cavaco, E., Casas, J.R., Neves, L. (2013) Quantifying redundancy and robustness of structures. *Proceedings of IABSE Workshop on Safety, Failures and Robustness of Larges Structures, Helsinki, Finland.*

Ghosn, M. and Moses, F. (1998) *Redundancy in Highway Bridge Superstructures*. NCHRP Report 406. Transportation Research Board, Washington DC

Knoll, F., Vogel, T. (2009) *Design for robustness*, Structural Engineering Documents, IABSE, Zurich.

Liu W. D., Ghosn, M., MOSES, F., NEUENHOFFER, A. (2000). *Redundancy in Highway Bridge Substructures*. NCHRP Report 458, Transportation Research Board, Washington DC

Moore, M. E., Phares, P. M., Graybeal, B. A., Rolander, D. D., Washer, G. A., (2001) *Reliability of Visual Inspection for Highway Bridges.* (FHWA Report Nos. FHWA-RD-01-020 and FHWA-RD-01-21) Federal Highway Administration, U.S. Department of Transportation, McLean, VA.

Tenzera, D., Puz, G., Radic, J., (2012) Visual inspection in evaluation of bridge condition." *Gradevinar*. vol.64, No. 9, pp.717-726

Vergani, M. (2010) Modellazione del degrade di strutture in calcestruzzo armato soggette a corrosion. Degree thesis, Politecnico di Milano, Italy (In Italian).

Vidal, T., Castel, A., Francois, R. (2004) Analyzing crack width to predict corrosion in reinforced concrete. *Cement and Concrete Research*, vol. 34, 165-174.