Life-cycle System Performance of Bridges: a Robustness-based Approach

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ABSTRACT: Corrosion in reinforced concrete and steel structures is among the most damaging phenomena to structural safety and therefore one of the most demanding when facing the life-cycle analysis of such structures. However, the onset and progression of corrosion is highly uncertain. Because damage due to corrosion appears due to environmental aggressive agents which progress on time, it is of interest to develop suitable life-cycle measures of structural robustness with respect to a progressive deterioration of the structural system performance. In this paper, a robustness-based approach as presented in Cavaco et al. (2013) is used to quantify the effect of corrosion on safety and life-cycle maintenance. The level of corrosion is considered as an uncertain parameter. In this way, a probabilistic measure of the robustness, seen as the tolerance to damage, can provide a comprehensive description of the life-cycle system performance. Therefore, a robustness definition and the calculation of the corresponding robustness index are proposed in the paper and further applied to one reinforced concrete and one steel existing bridge to show the practical application and results when applied to real bridges. It is shown how the obtained robustness index for the bridges can be directly co-related to their life-cycle performance and also used to define the optimum intervention strategies.

1. INTRODUCTION

Infrastructure deterioration and aging are among the most challenging issues that most developed societies are facing nowadays, since substantial investment has been made in last 50 years in the development of new infrastructures but significantly less has been devoted to their maintenance. The number of infrastructures now reaching the design life-time is growing year after year as the needs for substitution, repair and maintenance.

Traditionally, management of structural infrastructures has been condition oriented, meaning that the repair and maintenance agendas have been defined giving priority to structures in worst condition and neglecting the ability of certain structural typologies to tolerate damage and deterioration better than others.

Tolerance to damage and deterioration has been defined by Cavaco et al. (2013), as synonymous of structural robustness being measured by the following index:

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

where $f(x)$ is a normalized measure of structural performance, defined according to design objectives and generally given by the ratio between performance of intact and damaged structures, respectively. d refers to measures of damage or deterioration phenomena, responsible for performance decreasing, and given by the ratio between existing, and maximum possible damage. The robustness index results in an average value of the damaged structure performance in relation to that intact, considering all possible damage scenarios. It ranges from 0 to 1, depending if damage has full or null impact on structural performance, respectively. Equation (1) reflects structural behavior only, in contrast with life-cycle and time-variant reliability based approaches, already proposed (Biondini and Frangopol, 2014), which tend to include the propagation mechanisms. The main idea of the index in equation (1) is to be dependent only on the relation between performance and damage in order to have relative measures of robustness to compare different structural types. One of its advantages is that it does not need and absolute quantification of robustness, but just a relative measure useful to compare and to prioritize maintenance actions. It has also been argued (Biondini and Frangopol, 2014) that integral measures of robustness should be avoided since they can provide only average indications over the lifetime and are not able to describe the actual level of structural robustness, making impossible to distinguish, among structures with similar average performance, those which tend to exhibit sudden performance loss and those with gradual performance decreasing. Again, the limits of any definition of robustness will depend on its intended application. In the present case, the definition in equation (1) is based on the tolerance to damage and the foreseen application is on the maintenance of the structure, not on the influence of local failures in the global safety, as found in the more traditional definition of robustness. As suggested by some authors, the definition of a specific robustness indicator depends on the objective, as it is difficult to define an optimal and unique indicator for different damage and collapse scenarios (Starossek and Haberland, 2011). It is simple to understand the importance of considering structural robustness, as defined by Cavaco et al. (2013) in the context of structural management. Clusters of robust structures tend to require less maintenance and considering robustness in the decision making process leads to an optimum use of available funds. Moreover repair of robust structures can be postponed over less robust structures, if both present similar condition.

Steel corrosion is among the most damaging phenomena, leading to fast deterioration and reduced life time of both reinforced concrete and steel structures. In this paper, robustness of a reinforced concrete bridge and a steel bridge are assessed and discussed using the methodology proposed by Cavaco et al. (2013).

2. TERCENAS BRIDGE, PORTUGAL, 1968

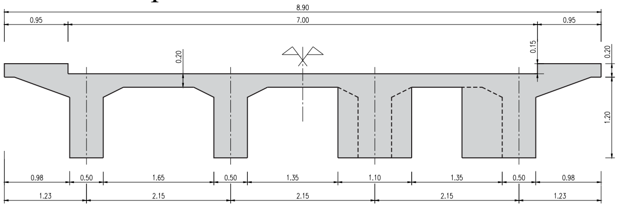
2.1. Introduction

Tercenas Bridge (see Figure 1) was built in Portugal in 1968 in Leiria district, providing access to Vieira beach. Due to its sea proximity, very earlier, the bridge started presenting signs of deterioration due to corrosion. These signs were aggravating year after year leading to the bridge demolition and substitution in 2012. Decision about bridge demolition was taken based mainly on its current condition in 2012 and concerns about the existing safety levels at the moment, neglecting the bridge robustness and its ability to tolerate corrosion. A significant investment was made in a new bridge, leaving less available funds to repair the remaining structures of the district.

Figure 1: Lateral view of Tercenas Bridge.

2.2. Bridge description

Structurally, the bridge was built in reinforced concrete with a total length of 60m, divided in three continuous spans of 18.60m, 22.80m and 18.60m. The bridge longitudinal axis crosses the river bed with a 19º angle. The three spans were supported by two abutments in the river margins and two piers in the riverbed. Due to the poor soil quality, all four vertical supports were founded on piles. The bridge deck, with 8.90m width, consisted on four girders connected on top by a concrete slab, giving place to the traffic lane and the sidewalks for the pedestrians (see Figure 2). The girders were 1.25m high and present linear variable width between 1.10m at the supports and 0.50m at the midspan. At each third of each span, 1.25m high and 0.30m wide crossbeams connected all the four girders, providing extra transversal stiffness to the bridge deck. Girders reinforcement was composed by a top single reinforcement layer of 16ϕ25 over the supports, and three bottom reinforcement layers of 5 ϕ 25, in the lateral spans, and one external layer of 6 ϕ 25, plus an internal layer of 5 ϕ 25 in the central span.



(a) (b)

Figure 2: Bridge deck cross section: (a) at mid-span and; (b) over supports.

2.3. Bridge Condition

Before its demolition in November 2012, the bridge had been regularly inspected since 2004, leading to the conclusion that the bridge condition was aggravating year after year. The most concerning part was found to be the bridge deck, in particular the bottom part of main girder 1, that on the sea side, therefore more exposed to salty winds, which was showing significant losses in most external reinforcement layer (see Figure 3). In 2010, it was concluded that girder 1 was heavily corroded all over its length, having lost 50% of effective reinforcement in most external layer. This resulted in approximately ¾ of effective reinforcement, as the remaining inner layer was found to be intact. Although it was not possible to check with accuracy corrosion on girders 2 to 4, these were considered much less corroded. No signs of corrosion were detected in top reinforcement as this was protected by the bituminous waterproofing layer of the traffic lane. Remaining structural parts (piers and abutments), although also deteriorated, were found less concerning.

Figure 3: Reinforcement corrosion on bridge deck main girder 1.

2.4. Bridge numerical model

Robustness analysis was limited to the bridge deck, the structural part which have raised more concerns. A tridimensional FE model of bridge deck was built in the OPENSEES software. Main girders, cross beams and top slab were modeled using nonlinear force based finite elements with distributed plasticity. Concrete compressive strength was considered equal to 51.2MPa, according to tests on extracted samples. Based on original design plans an A400 steel grade was adopted. Dead loads were also estimated considering original design plans. Since no traffic records were available for this bridge, current CEN (2002) prescriptions were observed. The 7.0m bridge lane was divided into three sub lanes, two with 3.0m width and the remaining one with 1.0m. The first lane was loaded with 9.0kN/m2. In the remaining lanes a uniform load of 2.5kN/m2 was applied. On the two first lanes two axle vehicles were considered with a distance of 2.0m between axles and wheels and with a 300kN and 200kN load/axle, respectively. The first and heaviest lane was defined preferentially over girder 1 (the most corroded) and both vehicles were positioned at the middle of the central span in order to account for the most adverse situation. Additionally, lateral span traffic lanes were unloaded, in order to maximize bending moment in the middle section of central span. It is important to refer that accordingly to the original plans the bridge was designed to sustain a three axle vehicle with a total weight of

450kN plus a dynamic factor of 1.2. These loads are clearly outdated when compared with the CEN (2002) prescriptions.

Most important deteriorating mechanisms resulting from reinforcement corrosion are: reinforcement area reduction, concrete cracking and spalling, bond strength reduction and ductility reduction of reinforcement (FIB 2000). This last phenomenon was neglected in this case, as it is of more concern when corrosion tends to be localized on specific spots, which was not the case, leading to localized strains and therefore to ductility reduction. To characterize bond strength reduction the M-pull model proposed by Bhargava et al. (2007) was considered as it is based on a significant set of experimental tests. The slipping fiber model proposed by Oliver et al. (2008) was used to model reinforcement plus its interface with surrounding concrete. Reinforcement area reduction was considered to affect spring related to steel fibers while bond strength reduction was considered to reduce spring strength related to the interface steel/concrete. The effect of cracking was considered in tension zones indirectly by neglecting tension strength of concrete. Compression zones were found to be not cracked.

2.5. Robustness assessment

In order to assess bridge deck robustness according to the methodology proposed by Cavaco et al. (2013), several analysis were carried out considering damage, in this case corrosion, ranging from zero up to the maximum observed during the bridge lifetime, 44 years.

Since corrosion was found to affect differently girder 1 and the remaining girders, five scenarios were considered. In the first case, corrosion level was considered equal among the four girders. Then, in girders 2 to 4, corrosion was decreased gradually, considering 75%, 50%, 25% and 0% of that affecting girder 1. In all cases, the numerical analysis was performed under imposed displacements, incrementing the partial safety factor, γ, applied to the traffic loads (both the uniform load and the two axes vehicles) and keeping dead load constant, until maximum load carrying capacity of the bridge deck was achieved. Figure 4 shows the maximum partial safety factor applied to the traffic loads as a function of normalized corrosion on girder 1 for the 5 different corrosion scenarios. Maximum normalized corrosion respect to 50% weight loss of steel bars (on the most external reinforcement layer) occurred during the bride lifetime.

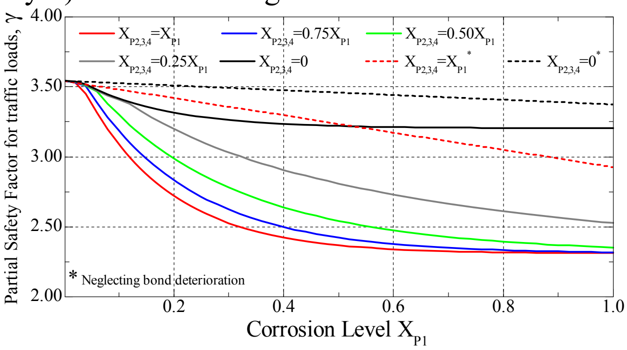


Figure 4. Safety factor for traffic loads as a function of the corrosion level on beam 1.

For the intact and uncorroded structure, the maximum safety factor resulted in 3.54, a relatively high value attending to the fact that the bridge was designed considering much lighter traffic loads, however obtained considering mean values for the material properties and for dead loads. For the worst case scenario, where corrosion was considered equal in all girders ($X_{p2,3,4}$=1.00 X_{p1}), the safety factor for traffic loads decreases at a higher rate for corrosion levels up to 30%, mainly due to bond reduction as observed in Figure 4 by comparing with results obtained for identical corrosion scenario but neglecting bond deterioration. From this stage onward, safety factor reduction was almost residual, as reinforcement was mainly disconnected from concrete, decreasing to a minimum value equal to 2.30. As shown in Figure 4, considering less corrosion on girders 2 to 4, resulted in less safety reduction due to the redundant behavior of the bridge deck, mainly provided by the cross beams and top slab, which allow load redistribution from more corroded and heavier loaded girder 1 to remaining girders. For the hypothetical scenario of corrosion affecting only girder 1 ($X_{p2,3,4}$=0.00X_{p1}), traffic loads safety factor reduction is minimal, from 3.54 to 3.20. Robustness assessment resulting from the normalized area bellow each curve in Figure 4 is summarized in Table 1:

Table 1: Robustness assessment.

Case	*Robustness Index*
$X_{P2,3,4}=X_{P1}$	*0.72*
$X_{P2,3,4}=0.75\ X_{P1}$	*0.73*
$X_{P2,3,4}=0.50\ X_{P1}$	*0.76*
$X_{P2,3,4}=0.25\ X_{P1}$	*0.82*
$X_{P2,3,4}=0$	*0.92*
$X_{P2,3,4}=X_{P1}{}^{*}$	*0.91*
$X_{P2,3,4}=0^{*}$	*0.98*

** neglecting bond deterioration effect*

For the worst case, robustness results in 0.72 contrasting with a robustness of 0.92 obtained if only girder 1 is affected by corrosion. These values represent the mean performance of the corroded structure (considering corrosion only on the first layer of reinforcement) in relation to the intact structure. Robustness of the bridge deck is increased due to redundancy, enhanced when less corrosion is considered on girders 2 to 4. As observed, by comparing in Table 1 robustness values considering and neglecting bond deterioration, this phenomena plays a fundamental role on the performance reduction of corroded structures, as suggested by Mangat and Elgarf (1999).

3. ABY BRIDGE, SWEDEN, 1951

3.1. Introduction

The second example describes the results of the robustness analysis of the Aby Bridge (North Sweden) and its ability to continue to carry loads beyond the elastic limit when the different members of the bridge are subjected to an environmental degradation process (corrosion). The object of the analysis is to investigate the residual capacity of the structure above the design load and taking into account the predicted degradation scenario, thanks to the inherent robustness and tolerance to damage as described by the robustness index.

3.2. Bridge description

Aby Bridge is a simply supported steel truss structure with a span-length of 33 m (Figure 5). The bridge is of the through type with two main longitudinal trusses connected by transverse beams at the floor level. Since it was built in 1951, it was designed according to the present trainloads type F46 which corresponds to 25 tons axle load which also is the present load on the railway. According to the design specification, the steel quality is of types S1311 for the stringer beams, verticals and diagonals and S1411 for the main truss and cross girders (Swedish specifications). The yielding strength is 240 and 270 MPa and the ultimate strength 360 and 430 MPa respectively. The bridge had to be removed and for this reason a test to failure was carried out. After the bridge was taken out of service it was put on temporary supports close to the tracks where the sensors were mounted. The force on the bridge was induced by two jacks where the jack is attached to a girder that distributes the load to four equally distributed point loads. In order to be able to archive the force needed to load the bridge to failure the jacks needed to be attached to solid rock.

Figure 5: Lateral view of Aby Bridge.

3.3. Bridge Condition

The atmospheric corrosion of the steel members will be considered as the damage scenario in equation (1). It is widely accepted that the long-term atmospheric corrosion of steel conforms to an equation of the form:

$$C = At^{B} \qquad (2)$$

where C is the loss of thickness in each side of the steel element exposed to atmospheric corrosion, t is the time and A and B are parameters. Equation (2) is the basis to obtain the corrosion profiles. The methodology used to obtain the corrosion profiles over time is presented in Kallias and Chryssanthopoulos (2014) and is based on a framework for the deterioration modelling of the coating-steel substrate system which is in line with the exposure classification recommendations of BS EN ISO 9223 (2012a). As presented in Chryssantopoulos (2013), the bridge members as defined in Figure 6 were identified for the Aby bridge. In Figure 6, element types 1 and 3 are box shaped, element type 2 is an open I section and element type 4 is a U shape section. Coefficients A and B in equation 2 were calculated. The values A =0.025 and B=0.575 were obtained for an environmental exposure C2.

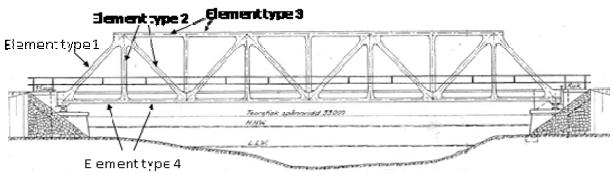


Figure 6: Aby Bridge element types.

According to BS EN ISO 9223, the C2 exposure corresponds to low corrosivity: Temperate zone, atmospheric environment with low pollution (SO2 < 5 μg/m3), e.g. rural areas, small towns. Dry or cold zone atmospheric environment with short time of wetness, e.g. deserts, subarctic areas. The performance profiles specific for that bridge, with and without paint coating, were obtained (MAINLINE-2014). As an example, in Figure 7 is shown the plate thickness evolution with time for the element type 3 in the cases with and without coating.

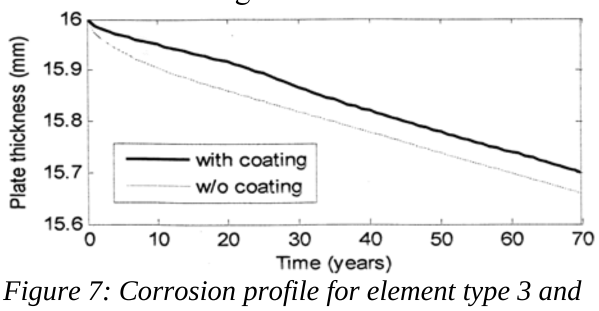


Figure 7: Corrosion profile for element type 3 and class C2.

3.4. Bridge numerical model

The numerical non-linear model of the bridge was developed using the ABAQUS software. The Finite Element Model (FEM) is made of shell elements taking all the connections as rigid.
The strain-stress relationship for the structural steel is considered as bi-linear with a hardening modulus (H) in its second part.
The following actions were considered in the analysis: self-weight of the structure, additional permanent loads and live load on the railway track including impact (UIC train load model) as described in Eurocode 1 (CEN-2002).

3.5. Robustness assessment

According to equation 1, to calculate the robustness index, it is necessary to obtain the bridge performance for different levels of corrosion degradation, from 0 (intact bridge) to 1 (maximum corrosion after 100 years of exposure without coating). Different corrosion scenarios have been considered, affecting elements O4, V5, U4, T5, V4, D4, D3, T4 alone in one longitudinal truss, as presented in Figure 8 (T are the transverse elements, not appearing in the Figure 8). Corrosion affecting all elements (CGEN) in both longitudinal carrying trusses was also considered.

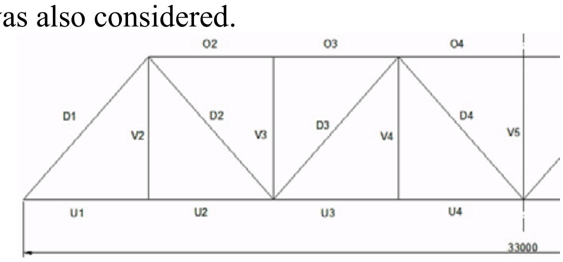


Figure 8: Description of corrosion scenarios

Figure 9 shows the normalized performance-normalized damaged curve in the case of exposure class C2. The performance indicator is the ultimate load (peak load when buckling of the compression upper chord occurs). The initial live load considered on the bridge corresponds to the UIC train load model (axle load (250 kN) and distributed load (80 kN/m). This basic load is incremented by a load factor until failure is achieved. The bridge has only 1 track. The case without coating is considered. In Figure 9, a normalized damage equal to 1 corresponds to a loss of thickness in steel plates corresponding to a service-life of 100 years. Different curves are drawn depending on the specific element where the corrosion is supposed to be effective (diagonals, upper chord etc.) to show the influence of this specific element and its corrosion on the bridge response.

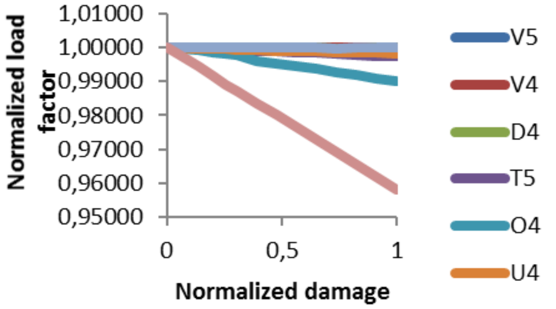
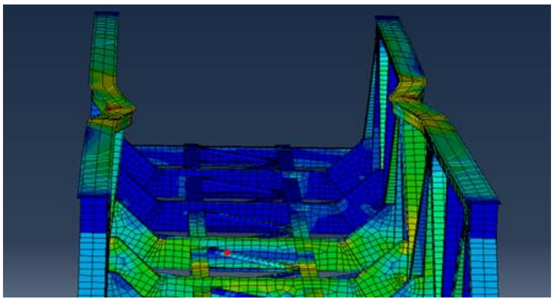


Figure 9: Normalized performance vs. normalized damage

Also the case of generalized corrosion in all elements (CGEN) is presented. As expected, the most sensitive member to corrosion is the upper chord, as the failure is due to buckling of this compressed element at mid-span (see Figure 9 and 10).



(a)

(b)

Figure 10: The global failure mode, buckling of the top frame in the truss: (a) numerical results vs. (b) experimental results.

The bridge is almost non-sensitive to the corrosion in other key members as diagonals and tension chord. Even in the case of corrosion in all members of the bridge, the load capacity only decreases around 4 % for the maximum service life. The robustness index obtained in this case is Rd = 0.98. The accuracy of the numerical model in predicting the failure mode can be assessed looking at Figure 10, showing the bridge after the execution of the load test up to failure after removal of the bridge from its original location.

The high value of robustness obtained indicates that the bridge is able to maintain the required safety level without any maintenance, or, in other words, the bridge has the ability to accommodate to degradation and can wait to maintenance interventions.

4. DISCUSSION

In the case of the truss steel railway bridge, the generalized corrosion of all elements derives a robustness index of 0.98 taking into account a damage scenario lasting over 100 years. This is a much higher value than the one obtained for the reinforced concrete bridge (even considering a damage scenario lasting over less than 100 years and corrosion affecting only girder 1) and is close to the maximum achievable value of 1. The bridge is, therefore, highly robust.

With only two main bearing longitudinal members supporting the deck, the steel truss bridge has a higher robustness, compared to the concrete bridge which has four main members. This result seems contradictory from the point of view of redundancy and may be due to the more aggressive environment present in the location of the reinforced concrete bridge and due to effect of bond deterioration which has significant impact on bridge performance. In fact, as shown in Figure 7, the loss of thickness in the plates of the bridge elements is very low, even in the case of no coating. In any case, this means that the truss bridge presents a higher ability to adapt to damage because of corrosion deterioration. The examples show how robustness to damage by corrosion and structural redundancy are not equivalent terms. The environmental conditions at the bridge site should be also considered when deciding on the best management policies.

5. CONCLUSIONS

The robustness index herein defined is a good indicator regarding the future performance of bridges and therefore is a useful tool looking to the best management decisions to be adopted when managing a set of different bridge types. The index combines the effects of structural type and environment conditions. In this sense, less redundant structural types not always are the main candidates to be strengthened or replaced, depending on the ability to tolerate damage. Again, a high value of the robustness index indicates that postponing interventions due to lack of funding is less critical. Comparing the results from 2 bridges analyzed in this work, the conclusion is that in the case of lack of funding to carry out the necessary intervention policies in both bridges at the same time, the highest priority should be given to the bridge with lower robustness index. This conclusion can be applied not only to a specific bridge, but also to a group of bridges with similar structural configuration and surrounding environments. As a result, similar planned interventions in sufficiently enough robust bridges can be postponed and grouped together to get a more optimal allocation of maintenance resources.

6. ACKNOWLEDGMENTS

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