Performance profiles of metallic bridges subject to coating degradation and atmospheric corrosion

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This study focuses on deterioration modelling and performance assessment of metallic bridges affected by atmospheric corrosion, considering also the contribution of typical protective systems in the form of multi-layer coatings. The mechanisms leading to coating degradation are reviewed and the main coating types used by infrastructure owners are highlighted. Building on information contained in industry manuals, a simple model for coating degradation is proposed. Atmospheric corrosion models are then presented, with emphasis given to exposure classification, in line with corrosivity classification guidelines and recent research quantifying the influence of corrosion through dose response functions. Coating degradation and corrosion models are integrated into a modelling framework, aimed at producing performance profiles of elements in metallic railway bridges. Finally, the framework is implemented in a case study in which a range of condition and resistance performance criteria are presented for different elements, such as girders or stiffeners, and their constituent parts, such as webs and flanges. It is shown that the proposed methodology is sufficiently detailed to enable differentiated performance predictions based on key external factors, and has scope for improvement, especially as coating and corrosion models are informed by the collection of field data.

keywords: performance profiles, corrosion, coatings, degradation, metallic bridges.

Nomenclature

A is an empirical constant depending on exposure conditions (mm/year)

A_0 is the uncorroded area

$A(t)$ is the corroded area

$A_{pr}(t)$ is the residual protected area of coated surface

A_{pr0} is the initial protected area of coated surface

B is an empirical constant depending on exposure conditions

$C(t)$ is the uniform thickness loss (mm)

Cl is the (annual average) chloride deposition rate (mg/m².d)

- 53 f_{St} is a function of T
- 54 I_0 is the uncorroded 2nd moment of area about the relevant member axis (major/minor).
- 55 $I(t)$ is the corroded 2nd moment of area about the relevant member axis (major/minor).
- 56 $R_A(t)$ is the buckling ratio
- 57 $R_B(t)$ is the bending resistance ratio
- 58 $R_{EB}(t)$ is the elastic buckling ratio
- 59 RH is the average annual relative humidity (%)
- 60 $R_{LB}(t)$ is the local buckling ratio
- 61 $R_S(t)$ is the shear resistance ratio
- 62 S_0 is the initial (uncorroded) value of section modulus
- 63 SO is the average annual deposition of SO_2 (mg/m².d)
- 64 SO_2 is the sulphur dioxide concentration (µg/m³)
- 65 $S(t)$ is the time-varying (corroded) value of section modulus
- 66 t is the time (years)
- 67 T is the (annual average) air temperature (°C)
- 68 $t_{fl,0}$ is the uncorroded thicknesses of the compression flange (or any other element)
- 69 $t_{fl}(t)$ is the corroded thicknesses of the compression flange (or any other element)
- 70 T_L is the service life
- 71 TOW is the time of wetness (h/year)
- 72 T_U is the time at which the coating is lost from the entire surface area
- 73 t_{w0} is the thickness of the uncorroded web element
- 74 $t_w(t)$ is the thickness of the corroded web element
- 75

1 Introduction

According to an extensive demographic survey undertaken for the Sustainable Bridges project (Bell, 2007), more than 35% of European railway bridges - whose total number is estimated to be well in excess of 300,000 - are over 100 years old. Although bridges of masonry construction are the single largest group within the overall population, metallic bridges – including those with steel/concrete or encased beam construction – comprise more than a third, with about one in five being centenarian. Steel and wrought iron are the predominant metals, with the latter having been used extensively in the 19th century until the era of steel began. In fact, more than 15,000 wrought iron bridges remain in service, mainly on the UK railway network, which had already expanded significantly by the time when steel was mass-produced. The same survey, which spanned railway authorities from Italy to Finland and from Poland to Ireland, revealed that the main asset management priorities relate to the improvement of the assessment process, with refined inspection/diagnosis tools and performance prediction also attracting significant attention.

Due to a large proportion of these bridges being on heavily utilized networks, large scale replacement is practically impossible, notwithstanding the worldwide financial constraints imposed on infrastructure maintenance budgets. Therefore, refined assessment methods and, where necessary, techniques that can improve prediction capability with respect to remaining service lives, thus allowing a life extension even by a modest fraction, are actively being sought. These refined methods could help in improving decision making related to asset management, and enable an orderly transition to the next generation of railway infrastructure. However, these decisions need to be taken in the context of aging and exposure to harsh environmental conditions, both of which contribute to wear and deterioration. In the case of metallic bridges, the former manifests itself through fatigue, whereas the latter is principally related to propensity to corrosion; either can have major

implications on safety, functionality and appearance. This study focuses on corrosion, which is a commonly observed cause of damage and failure in metallic bridges (Wardhana & Hadipriono, 2003, Imam & Chryssanthopoulos, 2012).

The occurrence and the subsequent rate of progression of the various types of corrosion depend on several factors and their interactions, with the majority associated with exposure conditions and the type of protection system applied, though maintenance practices also play a part. The progression of corrosion damage can lead to inadequate performance and reduced safety, which ultimately can result in structural failures. However, even before structural failure is reached, corrosion deterioration can be assessed as unacceptable on the basis of condition criteria. In simple terms, corrosion can be classified as general or local, though a more precise classification based on the forms of corrosion (e.g. Landolfo et al., 2010) subdivides it into general (uniform), pitting (localised), crevice, erosion, galvanic and fatigue corrosion. It should be noted that over time, a localised region can spread spatially or, conversely, a uniform domain can develop local patterns, emphasizing the time-variant complexities of the corrosion process.

Several studies have been carried out dealing with performance assessment of deteriorating steel bridges (e.g. Kayser & Nowak, 1989, Czarnecki & Nowak, 2008, Sharifi & Paik, 2011). However, a systematic exposure classification is hitherto lacking, and corrosion damage is predicted from models based on non-homogeneous databases, typically associated with unqualified levels of uncertainty. Furthermore, in most studies the time-dependent performance of the protective system, which needs to be broken down before corrosion damage occurs on the metallic surface, has not been considered explicitly.

This paper focuses on deterioration modelling and performance assessment of metallic bridges affected by atmospheric corrosion, considering also the contribution of typical protective systems. Emphasis is given to exposure classification, in line with

corrosivity classification guidelines and research aimed at quantifying the influence of corrosion through dose response functions. A distinction is made between condition and resistance based performance criteria, to suit various asset management objectives, which can be addressed through a hierarchical modelling process depending on available information. The proposed methodology is demonstrated using typical metallic bridge elements, for which performance profiles are developed under different short- and long-term exposure scenarios.

2 Deterioration modelling framework

2.1 Exposure classification

Coating deterioration and steel corrosion (following the loss of protection provided by the coating) are time-variant processes, with their rate being determined by the outdoor exposure conditions experienced by the bridge. They can be broadly classified as: (a) immersed, (b) splash zone and (c) atmospheric exposure. Each of these environments is associated with different ranges of corrosivity potential. This study focuses on atmospheric corrosion, which can itself be subdivided in a number of categories, namely rural, urban, industrial/coastal exposures (Figure 1). Several studies (e.g. Feliu et al., 1993, Gascoyne & Bottomley, 1995, Klinesmith et al., 2007, BS EN ISO, 2012a) have concluded that the main parameters influencing the atmosphere's corrosivity are climatic factors (i.e. relative humidity and temperature) and atmospheric pollutants (i.e. sulphur dioxide SO₂ and chlorides Cl⁻). Physical monitoring of these parameters could be used to determine the atmospheric corrosivity classification of a particular location.

The standard BS EN ISO 9223 (2012a) provides a framework for the classification of atmospheric corrosivity based on the levels of the main influencing climatic and pollutant variables. Specifically, in BS EN ISO 9223, the spectrum of atmospheric corrosivity is divided into five categories varying from very low (category C1), corresponding to rural environments, up to very high (C5) and extreme corrosivity (CX). In turn, the correlation of

these exposure classifiers with metal loss measurements has allowed the development of corrosion models (discussed in section 2.3). Table 1 briefly describes each corrosivity category together with the expected ranges of corrosion rates within each category.

In considering the deterioration of complex and spatially extended structural systems such as bridges, individual elements (e.g. deck, main girders, cross beams, etc.) are likely to experience dissimilar corrosion rates due to differences in the microclimate which develops in their immediate surroundings (Hutchins & McKenzie, 1973). For instance, a Japanese survey has shown that external main girders of steel bridges are more susceptible to corrosion than inner girders (Tamakoshi et al., 2006).

2.2 Protective coatings

One of the aims of this study is to examine the performance of coatings applied onto metallic surfaces of bridge components (BS EN ISO 12944). Several coating types – of varying composition and performance – exist for the protection of structural steelwork, including organic, inorganic, metallic, duplex and hybrid systems (MAINLINE, 2014). In practice, the majority of the coating systems consist of several layers with each layer performing a different function. Organic, metallic and duplex (hybrid) coatings protect the substrate metal through the interface with the anodic and cathodic reactions in the cell and/or by hindering the transport of ions to the substrate surface. Based on their resistance mechanisms, coatings can be classified in the following groups (Greenfield & Scantlebury, 2000, Hare 2006, de Wit et al., 2011):

- Barrier coatings, which suppress the diffusion of ions through the coating to the substrate surface.
- Inhibitive coatings, which promote the formation of an insoluble passive film on the metal substrate; careful selection of the binder and the inhibitive pigment is required to

avoid the rapid exhaust of the inhibitive ions provided by the pigment and reduce blistering.

- Sacrificial coatings, which protect the substrate metal through the mechanism of galvanic corrosion; here, the metal powder in the coating becomes the anode while the substrate metal becomes the cathode.

2.2.1 Coating types

This section highlights the compositions and performance characteristics of the main coating types. Organic coating systems are built-up by several compatible layers, producing a wide range of properties by varying the composition and thickness of the individual layers. The main constituents of organic coating layers are the binder (usually an epoxy matrix), the pigment and the solvent, with the former determining the physical and chemical properties of the coating. Fillers and additives are commonly used to add specific properties to the final coating, such as impact and abrasion resistance, UV absorption, etc. (Keijman, 1999, de Wit et al., 2011). The first layer, placed in direct contact with the substrate metal, is the primer which can be enriched with corrosion inhibitive pigments, e.g. galvanic pigments incorporating metallic zinc particles (Hare, 2006). The function of the topcoat (or finish) layer is to provide initial resistance against the external environment and determine the aesthetic appearance (e.g. colour).

Metallic coatings provide excellent long-term corrosion protection to metallic structures subjected to a range of harsh exposure conditions (e.g. marine environments). These coatings protect the substrate metal through sacrificial cathodic protection together with barrier action. The metallic coating, typically zinc in hot-dip galvanizing or aluminium in thermal sprayed coatings, acts as the anode of an electrochemical cell (CORUS, 2004). In thermal spraying applications, a heated spray gun (oxygen flame or electric arc) is fed with the metals (powder or wire form) and the melted metal is blown onto the metallic surface

using a compressed air jet (CORUS, 2004). Suitable surface preparation facilitates the mechanical bond between the coating and the substrate metal. Guidance on the use of thermal spraying aluminium/zinc coatings can be found in BS EN 22063.

Duplex (metallic-organic layers) and hybrid (metallic-metallic layers) coating systems provide superior long-term corrosion protection. Typically, duplex coating systems consist of a metallic coating applied directly on the prepared substrate metal followed by subsequent organic coating layers. In this way, the pores of the metallic coating are sealed using the organic coating, adding a barrier against the ingress of corrosive species. In general, the main drawback of duplex and hybrid coating systems is their relatively higher initial cost; often this can be offset by the anticipated reduced need for both maintenance/repair and minimised service disruption. Hybrid coatings systems, consisting of metal alloys or two dissimilar metallic coating layers, have also been formulated. The results of Kuroda et al. (2006) from a long-term experimental program using thermally sprayed and hybrid Zinc-Aluminium alloy coatings showed that these coatings performed well in marine environments even when unsealed. Similar results were obtained by Salas et al. (2012) using a thermally sprayed two-layer Zinc-Aluminium hybrid coating system, with the Zinc layer applied as a primer.

2.2.2 Coating deterioration

Deterioration of in-service coatings is affected by several factors including coating specification, quality of the application (surface preparation, application, curing), the environmental exposure conditions and accidental damage. Clearly, the large number of variables, together with their possible interactions, makes coating deterioration a complex phenomenon. In general, the presence of defects within a coating system accelerates the deterioration process which eventually leads to coating failure. The most common failure mode is loss of adhesion, for instance due to blistering or cathodic delamination of the coating. Other coating failure criteria include the time needed for the consumption of the

active ingredients of inhibitive and sacrificial coatings (i.e. inhibitive pigment, consumption of zinc dust, etc.), though the actual time-to-failure is highly uncertain due to the variability in surface preparation, coating application quality, the constantly changing exposure conditions, inherent defects, as well as unexpected factors such as accidental damage. Further information on defects and failure modes of protective coatings can be found elsewhere (e.g. Greenfield & Scantlebury, 2000, Sorensen et al., 2009).

Cathodic delamination is a commonly observed failure mode of organic coatings caused by inherent or induced defects. Corrosion initiates at the defect, forming corrosion products which potentially block the pores. As a result, the area where the coating is damaged becomes the anode while cathodes develop at the edges of the defect. The alkaline environment ($\text{pH} > 12$ due to hydroxyl ions) at the cathodic areas of the substrate is responsible for bond loss at the coating-substrate interface, with the formation of insoluble corrosion products promoting the separation of anodes and cathodes. The rate controlling parameter in this failure mechanism is the diffusion rate of cations required for charge neutralization of the cathodically produced hydroxyl ions.

Blisters are typically associated with coatings where no defect is visible. The possible mechanisms involved in this type of failure include expansion due to swelling, gas inclusion or osmotic processes (Sorensen et al., 2009). Among these mechanisms, osmotic processes have been shown to be the most significant in promoting blistering. Greenfield & Scantlebury (2000) have studied the development of blisters, which have been classified as osmotic, anodic and cathodic.

2.2.3 Coating selection

In the United Kingdom, corrosion protection of railway bridges using coatings and sealants is covered by guidance documents produced by the infrastructure owner. Current recommendations distinguish between new and existing steelwork, with a number of coating

systems recommended for each category (see NR 2009a, b & c). Commonly used coatings for new construction include duplex systems in which the first of several layers is a metallic coating, for instance thermally sprayed zinc or aluminium. On the other hand, coating systems for maintenance of existing structures are broadly subdivided into those recommended for patch repairs (bitumen based) and those for complete re-application. These protective systems typically consist of several layers applied sequentially on the treated steel surface. As mentioned above, the performance of a coating system is influenced by several factors, including the composition and exposure conditions, as well as workmanship and quality control during application. For example, Table 2 summarises the characteristics and expected service life of two coatings used in railway maintenance. Guiding values for the expected service life for other coating systems currently used are given in the aforementioned manuals (NR 2009a, b and c). It is also recommended that the selection of a suitable protective system be related to the anticipated exposure conditions (e.g. classification of exposure conditions based on BS EN ISO 12944-2).

2.2.4 Coating deterioration modelling

In general, the development of models to predict coating performance requires direct or indirect consideration of the following parameters:

- The coating's chemical and physical characteristics, including its principal resistance mechanism (e.g. barrier, inhibitive or galvanic action).
- The anticipated exposure conditions, including levels of individual climatic and atmospheric variables for a particular location.
- Other aspects such as the quality of surface preparation, operator's skills and experience, and coating curing characteristics.

Thus, modelling of coating performance can be attempted at different levels of complexity and accuracy, as outlined below:

- In Level 1 models, either a single value or a range is available for the expected service life of a coating system under a particular environmental exposure; no quantitative information is available on the influence of individual climatic or atmospheric factors on coating performance. Such models may include, based on inspection data, the primary statistical properties of the coating's service life (i.e. mean and standard deviation), which could reflect not only the influence of different exposure conditions (i.e. rural, urban, etc.) but also the variability in workmanship (e.g. poor, good, etc.).
- Level 2 models are based on mathematical relationships between coating performance and a set of influencing variables (e.g. temperature, humidity, etc.) known as dose response functions (DRF). Where the statistical properties of the influencing factors are also available, the corresponding coating performance statistics may be inferred using expectation properties or Monte Carlo simulation.
- Level 3 models are based on theoretically underpinned analytical and numerical simulations. As opposed to the DRFs, the underlying formulations describe the mechanisms involved in the deterioration processes (Pommersheim et al., 1994, Nguyen et al., 1995). These models can improve fundamental understanding through the interpretation of experimental data. Currently, their use is limited to carefully executed laboratory studies.

In the following, a simple (Level 1) model for time-dependent coating performance is developed, using information available in Network Rail standards where a range of expected life (T_L) values are provided for a number of coatings used in the UK (see NR 2009a, b & c). Experimental results (Itoh & Kim, 2006, Kim & Itoh, 2007) suggest that the coating itself will gradually deteriorate with time and will eventually become completely ineffective. This

assumes that the application of the coating is carried out by competent coating contractors, and, hence, the possibility of rapid deterioration and/or premature peeling are minimised. Furthermore, the results of Itoh and Kim indicate that coating deterioration tends to be non-linear over time, as schematically depicted in Figure 2. This is further supported by a study on the quality of steel bridge coatings (Chang et al., 2000) in which non-linear deterioration curves similar to those shown in Figure 2 are proposed based on the relationship between warranty period and expected service life. Building on these idealisations, a polynomial relationship is proposed, assuming that service life (T_L) estimates quoted in industry manuals correspond to circa 50% of a coated surface being unprotected at time T_L :

$$\frac{A_{pr}(t)}{A_{pr0}} = 1 - \left(\frac{0.6t^2}{T_L^2} - \frac{0.1t}{T_L} \right) \quad (1)$$

where $A_{pr}(t)$ and A_{pr0} are the residual and initial protected areas respectively and t is the time in years. This equation may be used to predict the time T_U at which the coating is lost from the entire surface area. Note that $A_{pr}(t) \leq A_{pr0}$ for all t and $A_{pr}(t) = 0$ for $t > T_U$.

In contrast to a linear relationship (also shown in Figure 2), for which $T_U = 2T_L$, the proposed polynomial leads to $T_U = 1.38 T_L$. Given the observed trends in experimental and field studies, this is a first attempt at a coating performance model, which can be used to estimate deterioration over any given surface (e.g. a web or a flange of a plate girder), for situations where the deterioration is likely to become progressively more extended. It should be noted that the coefficients in Equation (1) could be adjusted to reflect alternative values regarding the percentage coating loss taken as the criterion estimating expected service life. In general, such a model should consider coating performance not only at a particular location but also its spatial characteristics. To enable this, information on whether uniform or localised corrosion is likely to be developed on a particular member should also be utilised.

2.3 Atmospheric corrosion models

At a specimen scale, the progression of atmospheric corrosion in any given environment can be modelled in terms of thickness loss over an exposed surface area. Over the years, models of varying complexity and accuracy have been developed, as summarised in Table 3. The proposed classification accords with that adopted for coating deterioration in section 2.2.4.

Among Level 1 models, the best known is the power model (Feliu et al., 1993; Haagenrud & Henriksen, 1996) given by:

$$C(t) = At^B \quad (2)$$

where, $C(t)$ is the uniform (measured as average over relatively small specimen surface areas) thickness loss (mm) after an exposure period of t years and coefficients A (mm/year) and B (unitless) are empirical constants, obtained using regression analysis on physical test results grouped according to different atmospheric exposure conditions. This implies that the dependency on the exposure conditions and material type (e.g. type of steel) is captured implicitly using suitable values for coefficients A and B , where A represents the corrosion loss at the end of the first year and B controls the rate of loss in subsequent years.

Recommended values for A and B are generally deemed to exhibit high uncertainty, partly as a result of databases in which exposure conditions were poorly defined/grouped. It has also been suggested that Equation (2) is suitable for periods up to 20 years; for $t > 20$ years, thickness loss may be calculated using the linear relationship presented below (BS EN ISO, 2012b).

$$C(t > 20) = A \left[20^B + B(20^{B-1})(t - 20) \right] \quad (3)$$

Figure 3 shows the thickness loss over time, for different corrosivity categories, predicted using Equations (2) and (3), with coefficients A and B chosen as either mid-range or upper bound values. The results indicate that for corrosivity categories C1 to C4 the predicted

corrosion losses are less than about 0.5mm over a 30 year period, if no protective system is applied. On the other hand, much higher losses (more than twofold) are predicted for corrosivity category C5. The range in the values of the empirical coefficients A and B , indicative of the inherent uncertainty, can also be seen to affect the predicted losses.

Level 2 models offer the opportunity to relate directly the environmental and atmospheric pollutant variables to the rate of corrosion, instead of pooling the influence of all factors on empirical model constants. With the availability of data pertaining to environmental variables and/or atmospheric pollutant concentrations increasing (partly as a result of raised concerns over air quality in many urban/industrial areas), these more advanced models could, in future, offer refined corrosion predictions, potentially also allowing changes in input variables over time to be accommodated.

An example of a relationship, which can be used to estimate coefficient A in Equation (2) as a function of several environmental and atmospheric pollutant variables, leading to a Level 2 corrosion model, is given in BS EN ISO (2012a):

$$A = 1.77SO^{0.52} \exp(0.02RH + f_{st}) + 0.102Cl^{0.62} \exp(0.033RH + 0.04T) \quad (4)$$

where SO is the average annual deposition of SO_2 (mg/m².d), RH is the average annual relative humidity (%), Cl is the average annual deposition of Cl^- (mg/m².d), T is the annual (average) temperature (°C) and f_{st} is a function of T .

An alternative Level 2 model has been presented by Klinesmith *et al.* (2007):

$$C(t) = A' \left(\frac{TOW}{C_1} \right)^D \left(1 + \frac{SO_2}{E} \right)^F \left(1 + \frac{Cl}{G} \right)^H e^{J(T+T_0)} t^B = A_{KL} t^B \quad (5)$$

where TOW is the time of wetness (h/year), SO_2 is the sulphur dioxide concentration (µg/m³), Cl is the chloride deposition rate (mg/m².d), $C_1 = 3800$ h/year (mean value), $E =$ mean of measured values of SO_2 , T is the air temperature (°C), and $T_0 = 20$ °C. For flat carbon steel specimens (Klinesmith *et al.*, 2007): $A' = 13.4$, $B = 0.98$, $D = 0.46$, $F = 0.62$, $H = 0.34$ and $J =$

0.016. Comparing Equations (4) and (5), it can be observed that both models retain the exponential term (t^B) but differ in the modelling approach for the corrosion loss in the first year (represented by the remaining part of the function).

Figures 4(a) to 4(d) presents a sensitivity analysis on the effect of different climatic and atmospheric variables on the long-term predictions of corrosion thickness losses calculated using Equation (5). For the variables involved, typical values are used, as suggested in BS EN ISO (2012a). As can be seen, airborne salinity and SO₂ concentration have the greatest impact on the long-term rate of corrosion.

As mentioned earlier, Level 2 models could also capture the potential effects of changes to exposure conditions over longer time horizons. These may be associated with initiatives to reduce atmospheric pollution in specific regions or related to effects brought about by climate change. For this purpose, equation (3), which is recommended for estimating losses beyond 20 years, is re-cast in the following form:

$$C(t > 20) = A \cdot 20^B + A \cdot B(20^{B-1})(t - 20) = C(t = 20) + \frac{dC(t)}{dt}(t - 20) \quad (6)$$

where the first term represents the thickness loss due to corrosion over 20 years and the second term is the thickness loss from 20 years onwards. As can be seen, the first term (accounting for corrosion up to $t = 20$) is of exponential form, whereas the second term (accounting for corrosion when $t > 20$) is linear with a slope equal to $dC(t)/dt = A \cdot B(20^{B-1})$. In the case where there is a change in exposure conditions at a time $t > 20$ years, the slope of the linear portion $dC(t)/dt$ can be modified accordingly, by defining a new A_i coefficient, to account for the new exposure conditions. Thus, for n changes of exposure conditions beyond $t = 20$ years, the following expression may be used:

$$C(t > 20) = C(t = 20) + \sum_{i=1}^n \frac{dC_i(t)}{dt} T_i \quad (7)$$

where T_i represents the duration over which each A_i is valid after the initial 20-year period.

Figure 5 shows the corrosion loss predictions for two scenarios assuming atmospheric pollution levels in a specific area are reduced, together with a no-change case. The first scenario assumes a sharp reduction in the SO₂ concentration at year 20 from 250 to 125 µg/m³, whereas in the second scenario this reduction takes place gradually between year 20 and 40. Both emission reduction scenarios can be seen to result in lower thickness losses, circa 1 to 1.5mm, compared to the no-change case. Note that the lines for reduced emission scenarios become parallel after year 40, since the pollutants converge to the same level.

3 Performance assessment

The consequences of deterioration on metallic bridge elements can vary from aesthetic and non-structural issues to progressive weakening and catastrophic failures (Prucz & Kulicki, 1998). In the context of maintenance regimes, the former are addressed through condition surveys, whereas the latter necessitate structural assessments. In this section, the coating and corrosion models presented above are, first, combined so as to provide predictions of coating and material loss under different exposure conditions (condition-based maintenance) and, secondly, integrated within strength formulations relevant to different limit states, such as tension, compression, bending and shear (capacity-based maintenance).

In general, environmental exposure leads initially to a breakdown and loss of coating over a growing number and extent of surface spots, and, while this process is continuing, corrosion also begins to take place in the least protected and more vulnerable spots. As previously mentioned, the progression of deterioration will be influenced significantly by the type of coating and substrate material, the presence of pollutants, bacteria and micro-climate effects regarding temperature, humidity, wind direction, exposure to sunlight etc. Hence, prediction of deterioration rates will be characterised by uncertainty and complexity at different scales. In this respect, the proposed modelling approach cannot predict accurately the deterioration of a single structure unless it is combined with inspection outcomes, which

is beyond the scope of the present study. Nevertheless, it can provide an insight into median trends that may be observed over time in cohorts of similar structures.

Figure 6 shows schematically the proposed modelling approach for performance assessment. The coating performance is captured via Equation (1), with the service life, T_L , estimated based on information on particular coatings, e.g. as found in maintenance manuals. In estimating T_L the factors shown in Figure 6(a), (specification, application quality, exposure) should, where possible, be taken into account. Material loss due to corrosion will then start at different points in time on the exposed surface, as shown in Figure 6(b); for this part, either Level 1 (Equation (2)) or Level 2 (Equations (4) or (5)) corrosion models may be used, depending on available information on exposure conditions.

The loss of thickness due to corrosion over any particular surface of a single member (e.g. web or flange plates in Figure 6(b)) will have an impact on the available cross-sectional area, which, in turn, influences other section properties relevant to different limit states ((bending, compression, etc.), e.g. the second moment of area or the radius of gyration. Thinning of the web or flange plate can also affect adversely local buckling strength, which is typically a function of the member slenderness, e.g. the flange half-width over thickness ratio. Moreover, if corrosion is allowed to progress considerably over an entire structure, overall load distribution may be affected, due to changes in members' axial and bending stiffness. Following the approach suggested by Prucz and Kulicki (1998), a number of residual resistance factors are proposed in the following, covering the principal limit states encountered in structural assessments of girder and truss bridges. Thus, for a bending limit state, this factor is given by

$$R_B(t) = \left[\frac{S(t)}{S_0} \right] \quad (8)$$

where, $R_B(t)$ is the bending resistance ratio, S_0 and $S(t)$ are the initial (uncorroded) and the time-varying (corroded) values of section modulus respectively. Depending on the class of

the section (i.e. whether the elastic or plastic bending capacity can be reached), either the elastic or plastic section modulus would have to be considered (BS EN 1993-1-1, 2005). Clearly, for the calculation of the section modulus at time t , the distribution of the induced corrosion damage across the section needs to be idealised; here, unless specific scenarios based on micro-climate effects are considered, the baseline assumption is that the loss of material is spread uniformly across the width of the flanges and along the web depth. However, it is possible to adapt this equation to reflect the situation (sometimes observed in real cases) where the bottom flange suffers more than the top flange due to local water entrapment, bird fouling, etc.

If shear capacity is of concern, the corresponding ratio is given by

$$R_S(t) = \left[\frac{t_w(t)}{t_{w0}} \right] \quad (9)$$

where $R_S(t)$ is the shear resistance ratio and t_{w0} and $t_w(t)$ are the thicknesses of the uncorroded and corroded web elements respectively.

Instabilities may be an issue either in the form of member (Euler) buckling or in the form of local buckling, e.g. in flange outstands or slender webs. For member buckling, two ratios are relevant, depending on whether the member falls in a stocky or slender category. For the former, the cross-sectional area becomes the controlling geometric property since material failure precedes buckling, thus the buckling ratio becomes

$$R_A(t) = \left[\frac{A(t)}{A_0} \right] \quad (10)$$

with A_0 and $A(t)$ being the uncorroded and corroded areas. For slender members in compression governed by elastic buckling, the ratio changes to

$$R_{EB}(t) = \left[\frac{I(t)}{I_0} \right] \quad (11)$$

where, I_0 and $I(t)$ are the 2nd moments of area about the relevant member axis (major/minor).

As for local buckling, the available resistance, as determined via plate buckling formulae (Timoshenko, 1964), is proportional to the square of the thickness, and therefore, an appropriate local buckling ratio is given by

$$R_{LB}(t) = \left[\frac{t_{fl}(t)}{t_{fl,0}} \right]^2 \quad (12)$$

where, $t_{fl,0}$ and $t_{fl}(t)$ are the thicknesses of the compression flange (or any other element).

Finally, it is worth noting that for members under tension, the capacity is once more governed by the cross-sectional area and the relevant ratio is, thus, given by Equation (10).

4 Case study

Following the performance modelling framework presented in the preceding section, performance profiles are presented for members of a short-span (half-through) railway bridge with a span of 9.6m, as shown in Figure 7, located in a heavily polluted industrial site (C5 corrosivity classification as per Table 2). The examined bridge, which is typical on the UK railway network, consists of different member types, including external/internal main girders, stringers and cross-beams. The external main girders have 13mm thick top and bottom flanges and 10mm thick webs, with the overall height of the section being 1220mm. The yield strength of the material is taken as $f_y = 300$ MPa. It is further assumed that a protective coating is applied initially but no coating re-application takes place during the examined 30-year maintenance planning window. However, the framework can also be used for an uncoated structure to focus on the effect of the coating on the structural performance, as well as to include the effect of coating re-application within a given maintenance period.

As often observed in inspections, the actual position of the bridge members (i.e. internal/external) can influence the aggressiveness of the micro-climate to which their metal surfaces are exposed. In one such scenario, the exposure conditions faced by internal members are less harsh compared to external members: thus, whereas the global exposure

classification matches the microclimate of external members, internal members (e.g. stringers) are exposed to micro-climates of lower aggressiveness. As mentioned previously, this assumption can be further refined, should particular factors prevail that may distinguish sub-elements of a girder, either in a vertical direction (e.g. top vs. bottom flange) or horizontally (e.g. end vs. middle sections). Table 4 lists the scenarios for which performance profiles are developed. The service life of the coatings (T_L) is taken as the mid-range of the values given in Table 2. In view of the bridge type, bending, shear and local buckling limit states are considered for different members. In particular, overall bending and shear are examined for the internal and external girders, whereas local buckling is assessed for the compression flange of the same members. For completeness, other members (e.g. cross-girders, longitudinal stiffeners) could also be included, bearing in mind the particular limit states that may govern their performance.

5 Results and discussion

In this section the results obtained from the case study which illustrate the methodology are presented and discussed in relation to the observed performance for different coating types, exposure conditions and location of different elements within a typical metallic railway bridge. Combining condition-based indicators (e.g. % area of coating breakdown) with the evolution of structural resistance can be a useful tool within the context of asset management. Specifically, understanding the effect of different exposure conditions on the expected evolution of condition and/or resistance over time would allow the optimisation of resource allocation and justify the prioritisation of future examinations and assessments, whilst maintaining the overall risk associated with a bridge portfolio at tolerable levels.

5.1 Condition assessment

5.1.1 Coating performance

Figure 8 shows the evolution of coating performance predicted using Equation (1) for two coating systems subjected to two different atmospheric exposure conditions. The results indicate that for all cases examined, an initial period exists during which the coating remains intact, irrespective of the exposure conditions. However, the higher performance coating M21 is associated with a much longer period during which it remains fully effective; in fact almost double the period estimated for the M27.4 coating. Moreover, the effect of different exposure conditions becomes quite significant beyond this initial post-application period. For example, for coating M27.4, it can be seen that when 50% of the substrate metal area is unprotected under C3 exposure conditions while only approx. 25% of the area remains protected when considering the C5 environment. Such results can be used to facilitate decision making within the context of long term maintenance and whole life cost assessment (MAINLINE, 2013c). To this end, the collection of field data from regular inspections would allow the improvement of coating deterioration models, whereas simultaneous recording of the geographical locations would facilitate estimation of statistical properties. At present, although inspection data is being routinely collected, there is no attempt to correlate these with atmospheric conditions or positioning data, which hinders model development.

5.1.2 Thickness losses

Figures 9(a) and 9(b) show the predicted flange thickness and web losses over time for external girders (EMG) subjected to C5 exposure conditions. In all cases, an initial period exists during which coatings are fully effective and no thickness loss is recorded. Thereafter, gradual breakdown of the coating leads to thickness loss over time with the rate of thickness loss being a function of several environmental and pollution-related variables (e.g. temperature, humidity, SO₂, Cl). As previously discussed, their effect is collectively

expressed through a particular set of A and B coefficients. In Figures 9(a) and 9(b), results are also shown for the case where no coating is applied to the exposed surfaces; in this case, the residual thickness of the flanges and web (or alternatively loss of thickness) follow a non-linear curve which gradually becomes linear for $t > 20$ years. Figures 10(a) and 10(b) show that similar results are obtained for the internal main girder element (IMG); however, the predicted reductions of thickness are of much smaller magnitude due to the less aggressive atmospheric environment (C3) considered relevant for these elements, which has a dual benefit: (a) longer coating protection and (b) smaller corrosion rates. The assumption differentiating member exposure in a particular structure was based on surveys indicating the higher susceptibility to atmospheric corrosion of external bridge members, in comparison to their internal counterparts (Tamakoshi et al., 2006). It is important to note that micro-climate effects (e.g. water ingress, inaccessible/hidden parts) can reverse this trend and reveal higher corrosion in internal parts.

5.2 Resistance assessment

Performance profiles of deteriorating structures can be evaluated with respect to different serviceability and ultimate limit state criteria, for instance increasing deflections, fatigue, bending resistance, shear resistance, local and global buckling resistances. In this paper, results are presented for the bending, shear and local buckling resistances of the different element types (e.g. EMG, IMG) considering their relative location on the bridge as well as different types of protective coatings.

Figure 11 shows a comparison of bending performance profiles obtained using Equation (8) for EMG and IMG elements considering two different coating types, with the no-coating case also shown. The results indicate that, although the bridge is in a high corrosivity environment, different exposure conditions at element level (due to their relative position on the bridge) cause the bending resistance of external girder (EMG) elements to

deteriorate at a much faster pace relative to the internal main girder (IMG) element. The shear performance profiles (through Equation (9)) in Figure 12 follow a similar trend to the bending results; although in this case, the reduction in shear resistance is slightly more severe. For example, the results in Figures 11 and 12 indicate that at the end of the 30 year maintenance window, the shear and bending resistances of EMG with coating M274 have reduced by more than 12% and 10%, respectively. Figure 13 shows a comparison of the results for the local buckling performance profiles calculated using Equation (12). These results indicate that the reduction in buckling resistance over time occurs at a much higher rate relatively to the bending and shear resistances. Overall, the results indicate that the deterioration of all performance metrics examined, is moderate for an initial 10 year period, irrespectively of the exposure conditions and coating type, with the reductions being practically within 5%. However, after this initial period, significant differences in performance are predicted for the examined scenarios. For all performance metrics, the results highlight the significance of the relative location of an element within a bridge, as well as the impact of the protective system, on the progression of deterioration. Similar trends were observed in the results for the stringer (ST) elements, which – as for the IMG elements - are classified as internal elements and are, thus, exposed to lower corrosivity (C3).

5.3 General remarks

As discussed earlier, decision making in asset management can be based on condition and/or resistance criteria, which often complement each other. To this end, it is not unlikely to encounter cases where the gradual loss of condition (e.g. due to the breakdown of the protective coating system) is not associated with loss of resistance, at least during the early stages of the deterioration process. As such, considering both types of performance metrics within a common framework can inform risk-based examination and assessment regimes (e.g. Zonta, Zandonini & Bortot, 2007, Barone, Frangopol & Soliman, 2014). Furthermore,

such a framework is an integral part of any life-cycle cost methodology (MAINLINE, 2013c).

Differences between the progression of deterioration in relation to condition and structural performance are often directly related to the exposure conditions, as well as the coating type and its quality. Often, the effect of microclimate associated with specific detailing or other characteristics of a structure can be a significant factor in the evolution of both the condition and structural performance of an element (van de Lindt & Ahlborn, 2005). In this paper, effects arising from the different atmospheric exposure conditions are only considered; the occurrence and evolution of defects associated with local microclimates caused by unsuitable detailing (e.g. water traps, elements buried in ballast, timber-metal interfaces, etc.) on a structure have not been considered (van de Lindt & Ahlborn, 2005, Ahn et al., 2013, Khurrama et al., 2014). In practice, it is possible to observe significant localised defects associated with such detailing on structures located in relatively benign exposure conditions and which are otherwise in reasonable condition. Notwithstanding, the results of the case study demonstrate that the proposed framework is sufficiently detailed to differentiate performance predictions based on key external factors and has room for improvement, especially as coating and corrosion models are informed by the collection of field data. Finally, it is relatively straightforward to introduce uncertainty modelling on a number of key variables and to estimate the ensuing statistical properties of the performance profiles, in terms of median or appropriate fractile values.

7 Conclusions

A methodology has been presented for the development of performance profiles for deteriorating elements in metallic bridges. The gradual breakdown of the protective coating and the effects of atmospheric corrosion have both been modelled, taking into account their

dependency on environmental exposure conditions and a variety of other factors. An approach for long-term changes in pollution or climatic variables has also been proposed. The methodology has been demonstrated through generation of performance profiles for a short-span metallic railway bridge, assumed to be located in a harsh environment. It is shown that it can account for a wide range of exposure conditions, and that it can be adapted so as to cater for micro-climate effects both at inter- and intra-element level. Depending on policy, budget and maintenance constraints, ageing metallic bridges may have to be managed either using condition or resistance criteria. The developed methodology has the flexibility to enable both to be examined, using relatively simple models which have been based on industry manuals and international guidance documents.

8 Acknowledgements

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Tables

Table 1: Description of atmospheric corrosivity categories and corresponding corrosion rates (BS EN ISO 2012a).

Corrosivity category	Description	Corrosion rates ¹ (mm/year)
C1	Very low corrosivity: Dry or cold zone, atmospheric environment with very low pollution and time of wetness, e.g. certain deserts, Central Arctic/Antarctica.	≤ 0.0013
C2	Low corrosivity: Temperate zone, atmospheric environment with low pollution ($\text{SO}_2 < 5 \mu\text{g}/\text{m}^3$), e.g. rural areas, small towns. Dry or cold zone, atmospheric environment with short time of wetness, e.g. deserts, subarctic areas.	$0.0013 < A \leq 0.025$
C3	Medium corrosivity: Temperate zone, atmospheric environment with medium pollution ($\text{SO}_2: 5 \mu\text{g}/\text{m}^2$ to $30 \mu\text{g}/\text{m}^3$) or some effect of chlorides, e.g. urban areas, coastal areas with low deposition of chlorides. Subtropical and tropical zone, atmosphere with low pollution.	$0.025 < A \leq 0.050$
C4	High corrosivity: Temperate zone, atmospheric environment with high pollution ($\text{SO}_2: 30 \mu\text{g}/\text{m}^3$ to $90 \mu\text{g}/\text{m}^3$) or substantial effect of chlorides, e.g. polluted urban areas, industrial areas, coastal areas without spray of salt water or, exposure to effect of de-icing salts. Subtropical and tropical zone, atmosphere with medium pollution.	$0.050 < A \leq 0.080$
C5	Very high corrosivity: Temperate and subtropical zone, atmospheric environment with very high pollution ($\text{SO}_2: 90 \mu\text{g}/\text{m}^3$ to $250 \mu\text{g}/\text{m}^3$) and/or significant effect of chlorides, e.g. industrial areas, coastal areas, sheltered positions on coastline.	$0.080 < A \leq 0.200$
CX	Extreme corrosivity: Subtropical and tropical zone (very high time of wetness), atmospheric environment with high SO_2 pollution (higher than $250 \mu\text{g}/\text{m}^3$) including accompanying and production factors and/or strong effect of chlorides, e.g. extreme industrial areas, coastal and offshore areas, occasional contact with salt spray.	$0.200 < A \leq 0.700$

Notes: ¹ Corrosion rates correspond to the first year of exposure.

767 Table 2. Description and expected service life of protective coatings (NR 2009a and b).

Coating system name	Description [for more details see (NR 2009a, b)]	Expected life, T_L (years)
M27.4	<i>Protective system using bitumen.</i> 1 st layer: surface tolerant epoxy primer (min dft 100µm. Intermediate coat: gelled bituminous solution – aluminium tinted (min dft 200µm). Topcoat: gelled bituminous solution – black finish with min dft 200µm.	5-7
M21	<i>Coating using epoxy glass flake.</i> 1 st layer: epoxy blast primer with min dft 25µm. Intermediate layer: epoxy glass flake intermediate coat with min dft 40µm and for layer C select among the following (min dft 50µm): anti-graffiti paint or polyurethane coloured finish or acrylic urethane topcoat or polysiloxane topcoat.	18-22

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Table 3. Corrosion model classification.

Level	Description	Comments
1	Empirical models. Effect of all influencing factors taken into account through model constants	Model coefficients are associated with very high uncertainty. Statistical properties may not be reliable due to inhomogeneous samples. Questionable model transferability.
2	Empirical models which relate the rate of corrosion to specific exposure variables, also known as dose response functions (DRF).	Reliance on spatial data for atmospheric pollutants and climatic parameters. Potentially suitable for probabilistic analysis, though uncertainty modelling largely untested.
3	Theoretical models involving simulation techniques to predict airflow patterns and pollutant mass transfer on exposed surfaces.	Heavy reliance on modelling assumptions. Complex uncertainty modelling, currently lack of input data at desired granularity level.

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Table 4. Scenarios considered in the case study

Element type ¹	Exposure classification ²		Coating	
	Bridge level	Element level	Coating type	Service life ³ , T _L (years)
IMG	C5	C3	No Coating	-
IMG	C5	C3	M27.4	6
IMG	C5	C3	M21	20
EMG	C5	C5	No Coating	-
EMG	C5	C5	M27.4	5
EMG	C5	C5	M21	18
ST	C5	C3	No Coating	-
ST	C5	C3	M27.4	6
ST	C5	C3	M21	20

Notes: ¹ IMG = internal main girder, EMG = external main girder, ST = stringer beam, ² Environmental classification in line with BS EN ISO 9223, ³ Expected coating life based on NR/GN/CIV/002. Coating specifications are given in Table 2.

Figures

Figure 1

Figure 2

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Figure 3

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Figure 4

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Figure 5

Figure 6

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1102 **Figure captions**

1103 Fig. 1: Outdoor exposure environments (Sorensen et al., 2009).

1104 Fig. 2: Level 1 deterioration model for coating system performance.

1105 Fig. 3: Thickness loss over time predicted using Equations 2 and 3 and Table 1; (a) mid-range
1106 values for A; $B = 0.523$, (b) upper limits for A; $B = 0.523$ and (c) upper limits for A; B
1107 $= 0.575$.

1108 Fig. 4: Effect of varying climatic and atmospheric pollutant parameters on the corrosion
1109 losses with time, predicted using Equation (5); (a) time-of-wetness (TOW), (b) Cl
1110 deposition rate, (c) SO_2 concentration and (d) temperature.

1111 Fig. 5: Thickness loss over time under changing exposure conditions.

1112 Fig. 6: Performance modelling framework (a) factors under consideration (b) progression of
1113 deterioration on coated metallic surfaces.

1114 Fig. 7: Schematic view of the short-span railway bridge.

1115 Fig. 8: Performance profiles of two coating systems under different exposure conditions.

1116 Fig. 9: Residual thickness profiles for EMG elements under C5 exposure conditions: (a)
1117 flanges thickness and (b) web thickness.

1118 Fig. 10: Residual thickness profiles for IMG elements under C3 exposure conditions: (a)
1119 flanges thickness and (b) web thickness.

1120 Fig. 11: Bending resistance profiles for EMG and IMG elements under, respectively, C5 and
1121 C3 exposure conditions.

1122 Fig. 12: Shear resistance profiles for EMG and IMG elements under, respectively, C5 and C3
1123 exposure conditions.

1124 Fig. 13: Local buckling resistance profiles for EMG and IMG elements under, respectively,
1125 C5 and C3 exposure conditions.